SEPTEMBER 1959
No. 3
V. 31

JOURNAL
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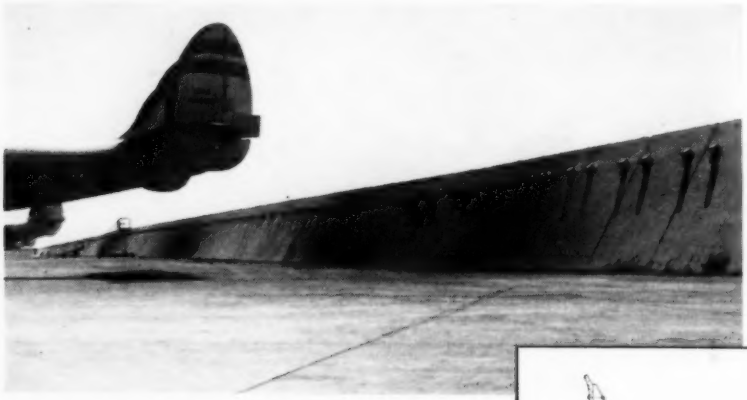
ACI
PROCEEDINGS
V. 56

Precast Blast Fences

. . . 'INCOR" ANSWER TO AN AIRPORT PROBLEM



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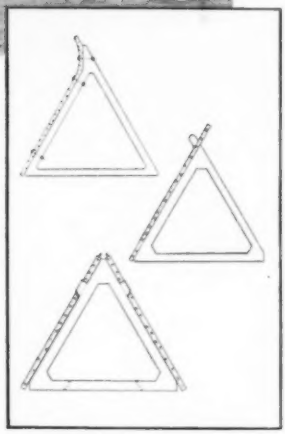

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VICTOR LEABU discusses "Problems and Performance of Precast Wall Panels" citing the need for sound engineering the development and use of precast concrete wall panels.

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YASUO KONDO, AKIHIKO TAKEDA, and SETSUJI HIDESHIMA describe experimental research conducted to investigate the "Effect of Admixtures on Electrolytic Corrosion of Steel Bars in Reinforced Concrete."

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J. GLUCKLICH comments on a set of simple static experiments conducted to determine the "Rheological Behavior of Hardened Cement Paste Under Low Stresses."

On the Cover—Chicago's Executive House. This 39-story, 371 ft high apartment house is built of high strength concrete and is supported on drilled caissons extending into bedrock. The story of this building appears on p. 215.

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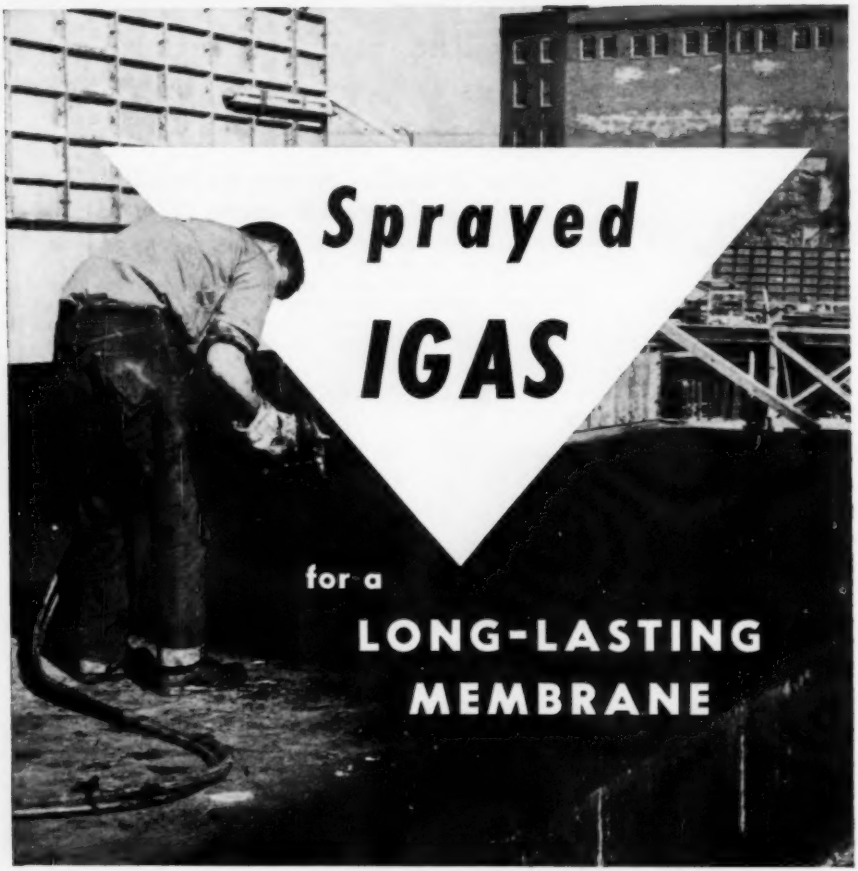

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LATERAL STABILITY OF REINFORCED CONCRETE BEAMS

By WILLIAM HANSELL and GEORGE WINTER

Some concrete design specifications, including the ACI Code, in various ways limit the distance between lateral supports of beams, presumably to safeguard against lateral buckling. The present investigation is intended to furnish some factual information on which to base such provisions. Ten tests on deep narrow beams have been carried out with unbraced lengths ranging from 28.8 to 86.4 times the beam width. No reduction in strength was observed over this range, showing the absence of lateral buckling. A tentative theory of lateral instability of reinforced concrete beams, including the effects of inelasticity and cracking, is given. It agrees with the tests in showing that present Code provisions are too restrictive, particularly for ordinary steel strengths. Theory indicates that closer lateral supports are required for high strength reinforced beams than for ordinary strength reinforcement.

■ DESIGN SPECIFICATIONS FOR STEEL STRUCTURES contain provisions for safeguarding unbraced beams against lateral buckling. These provisions are usually expressed in terms of slenderness ratios such as L/b or Ld/bt. Lateral buckling of beams, as shown in Fig. 1, involves both lateral bending and torsion. Since torsional rigidity is proportional to the cube of the thickness of a member, it is evident that this rigidity is relatively low for steel members with their comparatively small thicknesses of webs and flanges. It is for this reason that such members, if sufficiently slender, are liable to buckle laterally, and that appropriate measures must be taken in design to prevent such buckling.

In contrast, rectangular beams, such as occur in reinforced concrete structures, possess high torsional rigidity. This would indicate that the danger of lateral buckling is considerably less for such beams than for structural steel shapes. Nevertheless some concrete design codes contain provisions which restrict L/b-ratios, presumably for the same purpose of preventing lateral buckling.

It is the aim of this paper to investigate the lateral buckling of rectangular reinforced concrete beams to furnish some basis for the development of appropriate design provisions. Since the complex nature of reinforced concrete (inhomogeneity, partial cracking, limited elasticity, etc.) makes an entirely

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GEORGE WINTER, member of ACI Committee 318 and chairman of its subcommittee on ultimate strength design and member of several other important ACI committees, is professor and head of department of structural engineering at Cornell University, Ithaca, N. Y. He has published some 50 research papers in the course of his work and coauthored with M. Pei the first American paper on the design of folded (hipped) plate construction (ACI JOURNAL, January 1947). He is also coauthor with Urquhart and O'Rourke of "Design of Concrete Structures," and was the first recipient of the ASCE Moisseiff Award.

analytical treatment impossible, resort was had to tests of ten beams, with L/b ratios ranging from 28.8 to 86.4. It was the purpose of these tests to determine the manner in which such large L/b ratios affect flexural capacity. The paper reports the results of these tests, develops an approximate and somewhat tentative buckling formula for unbraced rectangular reinforced concrete beams, and draws conclusions in regard to the design of such members.

Notation

b = width of a rectangular beam
c = distance from the neutral axis of a concrete beam to the extreme compression fiber
d = depth of a concrete beam from center of tension reinforcement to extreme compression fiber
E = modulus of elasticity
E_c = initial tangent modulus of concrete
E_s = modulus of elasticity for steel
$E_{secc} = f_c/\epsilon_c$ = secant modulus of concrete pertaining to flexural stress-strain properties and extreme compression fiber stress and strain
E_t = tangent modulus
$\overline{E}$ = reduced modulus, used in inelastic buckling equations
f_c = concrete stress
f_c' = ultimate compressive strength of concrete at 28 days
G = modulus of elasticity in shear
$\overline{G}$ = reduced modulus, used in inelastic buckling equations
h = depth of a homogeneous beam
I_y = moment of inertia of a beam about the minor (vertical) axis of symmetry
K_t = torsional constant of a beam
$k = c/d$ = ratio indicating position of neutral axis
K_3 = ratio of ultimate flexural stress to ultimate compressive stress of concrete
L = distance between points of lateral support
M = bending moment
M_{cr} = critical buckling moment
M_{calc} = predicted ultimate moment
M_{test} = maximum moment observed in beam test
$w(y)$ = width of a beam at a distance y from the neutral axis
y = vertical distance from neutral axis
β, γ = parameters related to the shape of the flexural stress-strain relation of concrete (see Fig. 9)
ϵ = strain
ϵ_c = extreme compression fiber strain in concrete
ϵ_h = extreme fiber strain in homogeneous beam
ϵ_s = strain in tension steel
ϵ_{sy} = strain at yield point of steel
ϵ_o = concrete strain at ultimate stress
ϵ_u = ultimate concrete strain
μ = Poisson's ratio corresponding to elastic values of E and G
σ = stress in homogeneous beam
σ_h = extreme fiber stress in homogeneous beam

BACKGROUND

Little information is available in English literature concerning the lateral stability of reinforced concrete beams. No published reports of lateral instability failures are known to the authors. One experimental study involving tests of three rectangular concrete beams with a constant L/b ratio of 36 is reported.[1] This reference does not indicate whether the beams were restrained from lateral or rotational movements at the load point. Since such restraint has a considerable effect on the stability of slender beams, there is some question concerning conclusions drawn from these test results. Reference 2 gives the results of an analytical study of the stability problem for concrete beams. The author considers both the lateral and torsional rigidities of a beam and concludes that stability regulations which consider only the L/b ratio are basically unsound. However, this brief analysis assumes "uncracked," elastic, homogeneous beams with flexural strength defined by the straight-line theory. The question of whether or not this analysis is conservative, particularly at loads approaching ultimate, is a matter of conjecture since a concrete beam is cracked, inelastic, and nonhomogeneous.

The 1956 edition of the ACI Building Code specifies in Section 704 that: "The clear distance between lateral supports of a beam shall not exceed 32 times the least width of compression flange." It is presumed that this provision was intended to safeguard against lateral instability failures. There is reason to question whether this regulation is appropriate for its presumed purpose, since it is supported by little or no valid factual evidence. It is obvious from experience that the $L/b = 32$ limitation is safe, at least for construction materials and methods of long standing. It is not obvious as to whether this limitation is a necessary and economical solution to the lateral stability problem in reinforced concrete.

A rectangular, homogeneous, elastic beam in pure bending will buckle laterally at the bending moment[3]

$$M_{cr} = \frac{\pi}{L}\sqrt{E\,I_y\,G\,K_t} \qquad (1)$$

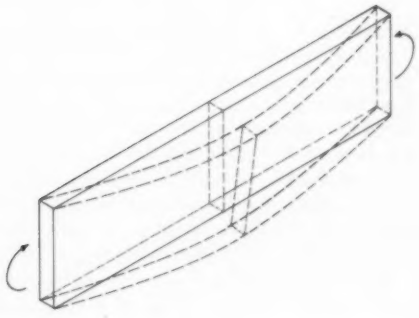

Fig. 1—Lateral buckling

TABLE 1—TEST BEAM DATA

Beam mark	Span, ft	L/b ratio	Stirrup spacing,* in.	Cylinder strength,† f_c', psi
B6	6	28.8	5½	4310
B9	9	43.2	5½	4310
B12	12	57.6	7	4350
B15	15	72.0	7	4215
B18	18	86.4	7	4260

*Uniform along entire span.
†Average of six 6 x 12-in. cylinders tested at 28 to 32 days.

For deep and narrow beams, with sufficient accuracy, $K_t = b^3h/3$. Using this value, the maximum fiber stress in such a beam at buckling is easily computed to be

$$\sigma_{cr} = \frac{\pi \sqrt{E\,G}}{(L/b)} \sqrt{\frac{I_y}{I_z}} \qquad \text{(1a)}$$

Two things are apparent from this equation: The stress at which buckling occurs decreases with increasing slenderness L/b, and this is the reason why design codes have often been formulated in these terms. It is also seen, however, that beams are the more unstable the smaller I_y/I_z, i.e., the narrower and deeper they are. This equally important fact is not reflected in formulas which merely contain L/b.

These simple equations cannot be applied directly to reinforced concrete beams in view of the complex character of the material. For this reason the test program reported below was carried out.

SPECIMENS AND TEST PROCEDURE

Specimens

The experimental program involved the testing to destruction of ten rectangular reinforced concrete beams of identical cross section, main reinforcement, and concrete mix. The beams were loaded at the quarter points on five simple spans of 6, 9, 12, 15, and 18 ft. Two companion beams were tested on each span. Beam specimens are designated by the letter B and two numbers. The first number is the simple span in feet and the second denotes one of two companion beams. All beams were 13 in. deep, 2½ in. wide, and used the same tensile reinforcement, one ¾-in. diameter deformed bar 11¼ in. from the extreme compression fiber. These dimensions provided a steel ratio of 1.56 percent. The beam section was purposely made unusually deep and narrow, i.e., more conducive to lateral instability [see Eq. (1a)], than beams normally encountered in practice. This was done to afford every possibility for lateral instability failures. Data concerning the beam specimens and the testing arrangement are included in Table 1 and Fig. 2.

The L/b ratios of the test specimens varied from 28.8 for the 6-ft span to 86.4 for the 18-ft span. All beams except the 6-ft spans violated the requirements of the 1956 ACI Building Code by as much as 270 percent. The 6-ft spans were included in the program to experimentally verify the ultimate flexural capacity without lateral buckling.

The beams were designed to fail in the flexural tension mode if lateral instability did not precede flexural failure. Quarter-point loading was used to create constant moment over an

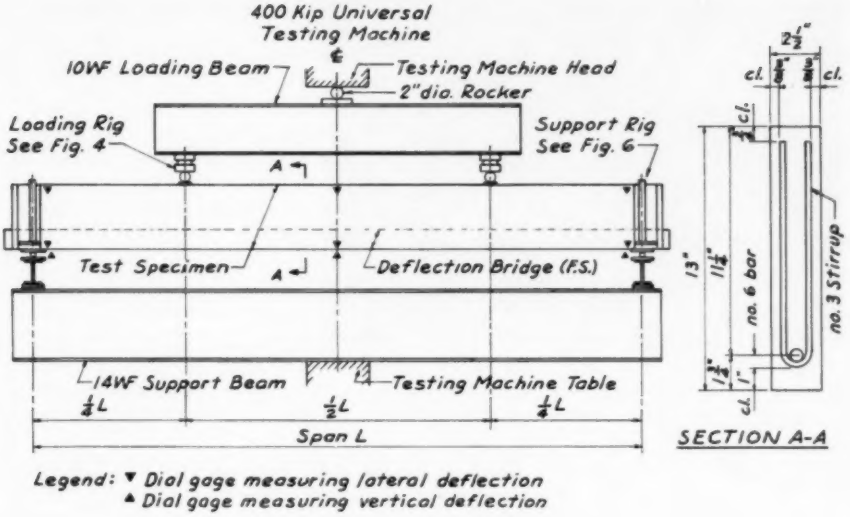

Fig. 2—Testing arrangement layout

appreciable portion of the span. As indicated in Reference 4, loading systems which produce near-maximum moments over sizeable portions of the span tend to increase the danger of lateral instability.* To minimize the possibility of shear failures, liberal web reinforcement was used along the entire span of all beams as indicated in Fig. 2 and Table 1.

Materials

Type I portland cement and intermediate grade deformed steel bars were used. Coarse aggregate was a graded gravel of ⅝-in. maximum size. Six 6 x 12-in. cylinders were cast and cured with each pair of test beams. Average cylinder test results are reported in Table 1. The average cylinder strength was 4290 psi.

The average yield stress and strain of the sharp yielding tensile reinforcement was 43,800 psi and 1.53 mills, respectively. These figures represent the average of four tension tests using standard 0.5-in. diameter specimens machined from the #6 bars.

Testing equipment

The beams were tested in a 400,000-lb capacity universal testing machine which was modified for the beam tests by bolting a steel "support" beam to the table of the machine and by suspending a steel "load" beam from the compression head of the machine. A diagram of the testing arrangement is shown in Fig. 2 and Fig. 3 shows a 12-ft test beam in the machine.

It was essential that the apparatus used to transmit load to the test beam (hereafter termed the loading rig) should provide a minimum of lateral and torsional restraint to the beam. Otherwise the test beam would have been restrained from the lateral and rotational movements associated with lateral instability. As indicated in Reference 5, mere friction may suffice to provide effective lateral bracing and prevent lateral buckling. Fig. 4 shows an assembly drawing of the loaded rig and Fig. 5 shows the rig in place during a beam test. The four rollers in the roller assembly are hollow automotive wrist pins. Keeper plates and roller pins maintain the rollers in a parallel configuration. The load ball is a 1½-in. diameter ball

*Although Reference 4 is primarily concerned with the lateral stability of steel I-beams, the comparison of critical buckling moments for various loading systems when the loads are applied at the centroid is at least qualitatively valid for reinforced concrete beams. See Eq. (48) of Reference 4.

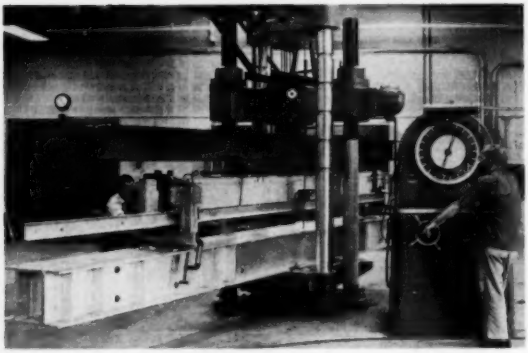

Fig. 3—Beam under test

bearing. All bearing surfaces of the loaded rig were hardened and ground smooth to reduce surface penetration and friction. All rolling surfaces were cleaned and lightly oiled before each test.

During the setup and centering phases of a beam test, the lateral bars were inserted in the positions shown in Fig. 4 (to impart stability), but were removed during the remainder of a test. The only lateral restraint offered by the loading rig (for lateral beam displacements of 1¼ in. or less at the load points) is developed by rolling friction at the roller assembly. Torsional restraint is limited to that developed by point contact between the load ball and

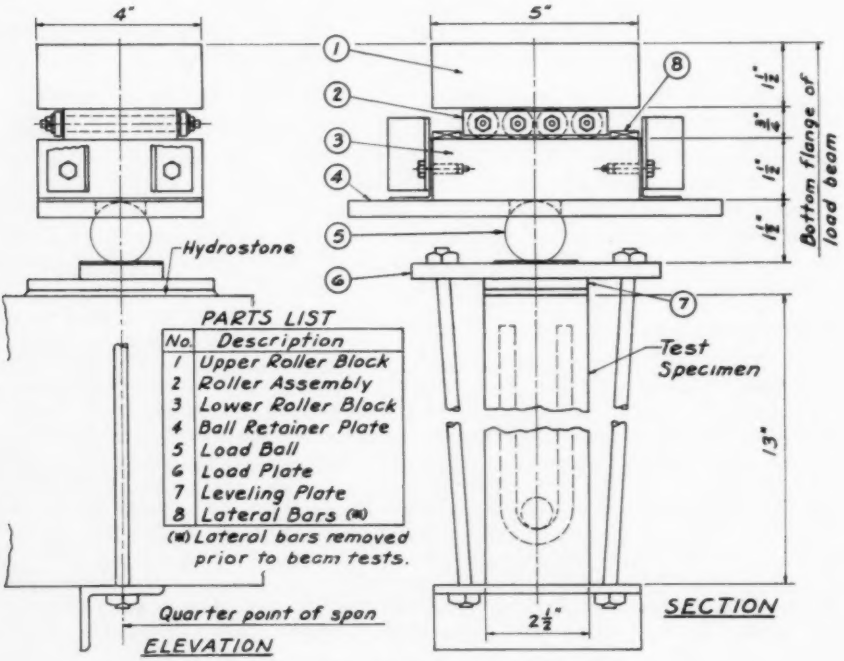

Fig. 4—Loading rig

the load plate. The line of action of the load passes through the point of contact between load ball and load plate and moves laterally with the beam.

Auxiliary tests* indicated that the maximum lateral restraining force developed by the loading rig did not exceed 0.1 percent of the vertical loads. Except for these negligible restraining forces the test beams were laterally and torsionally unrestrained over their entire simple span.

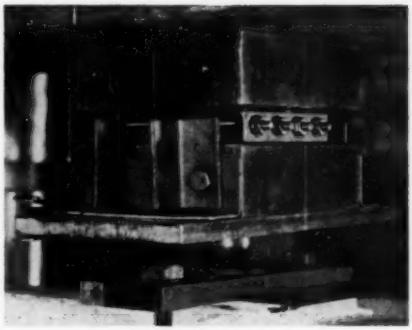

Fig. 5—Loading rig in place

The apparatus used to support a test beam (hereafter termed the support rig) permitted the beam to rotate about the principal axis of its cross section but prevented rotation about the longitudinal beam axis. This was accomplished as illustrated in Fig. 6 which also shows the idealized support conditions approximated by this rig. Fig. 3 shows the support rig in place during a beam test. Note the vertical rollers extending above the top of the test beam. Vertical and lateral deflections were measured using nine dial gages as indicated in Fig. 2. All dial gages (except the two measuring vertical movement at the supports) were clamped to a steel deflection bridge which was supported on the horizontal rollers at the ends of the simple span. Fig. 3 shows the deflection bridge with dial gages in place.

*These auxiliary tests are described in detail in Reference 6.

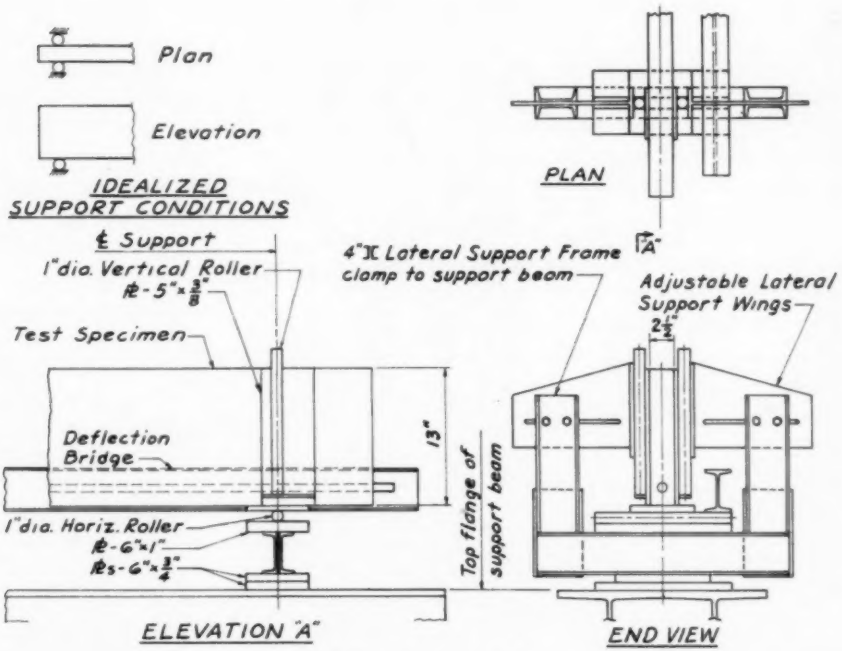

Fig. 6—Support rig

Testing procedure

The beams were measured with a steel scale and calipers and were found to be within ± 1/8 in. of nominal dimensions. A plumb bob, level, and measuring tape were used to accurately locate the beams in the testing machine.

Centering under load was accomplished by minimizing lateral deflection at midspan, under loads not exceeding one-third of the predicted ultimate load.

When the loading rig was centered, loads were applied in increments of about 10 percent of the predicted ultimate load up to 90 percent of this load. Thereafter, loads were applied in smaller increments at a reduced loading rate until it was evident from the rapid increase of vertical deflections that the flexural steel had yielded. The time between the first and last increment of load averaged about 1½ hr.

TEST RESULTS

Table 2 summarizes the results of the beam tests. In this table, M_{test} is the moment at midspan of the test beams (including dead load moment) corresponding to the maximum observed test load. The values of M_{calc} are the ultimate moments for simple flexural tension failure computed by using Eq. (A1) in the Appendix of the 1956 ACI Building Code. A flexural tension failure is initiated by yielding of the tensile steel. The marked deviation from approximately linear load-deflection behavior which occurs when yielding begins in an underreinforced beam defines the yield point of the beam. The yield point deflections of the test beams are entered in Table 2. This table also gives the initial lateral deflections measured prior to loading.

Inspection of the ratios M_{test}/M_{calc} in Table 2 indicates that slenderness or laterally unsupported length had no effect on the flexural capacity of the test beams. All ten beams in this experimental program failed in the flexural tension mode at practically identical moments, in close agreement with the predictions of ultimate strength theory. They were not weakened by lateral buckling. This was true in spite of the fact that the largest L/b ratio was

TABLE 2—RESULTS OF BEAM TESTS

Beam	B6	B9	B12	B15	B18	Suffix*
L/b ratio	28.8	43.2	57.6	72.0	86.4	1, 2
Observed ultimate moment, M_{test}, kip-in.	216 199	201 205	193 199	192 198	190 196	1 2
Calculated ultimate moment, M_{calc}, kip-in.	196.7	196.7	197.0	195.9	196.2	1, 2
M_{test}/M_{calc}	1.10 1.01	1.02 1.04	0.98 1.01	0.98 1.01	0.97 1.00	1 2
Midspan deflections at yield point, 0.001 in.						
Vertical	192 188	330 330	460 495	825 1005	1015 1080	1 2
Lateral-Top	53 78	33 18	43 515	1260 97	72 500	1 2
Lateral-Bottom	66 82	56 25	14 620	1090 228	150 430	1 2
Initial lateral deflection, 0.01 in.	4 6	8 6	8 11	25 12	13 17	1 2

*Suffix indicates one of two companion beams.

Fig. 7—Beam B18-2 after test

86.6 or three times the smallest, and 2.7 times the largest L/b ratio permitted by Section 704 of the 1956 ACI Building Code. The average value of the ratio M_{test}/M_{calc} was 1.01 indicating an excellent agreement between predicted and observed ultimate flexural capacities. The usual small variations in observed ultimate moments show no significant correlation with span lengths or lateral deflections. These are the most important results of the testing program.

One factor which may have had a minor influence on the results of the beam tests is the rather heavy web reinforcement used in the test specimens. The web reinforcement ratios were 1.60 percent for the 6- and 9-ft beams and 1.26 percent for the 12-, 15-, and 18-ft beams. This reinforcement undoubtedly contributed to the torsional strength of the beams and may have prevented potential torsional failures in those beams which developed large lateral deflections.

Fig. 7 shows Beam B18-2 after completion of its test. The crack pattern for this beam is typical of that for all of the test specimens. Several diagonal tension cracks, due to vertical shearing stresses, developed near the supports of the beams at loads approaching the yield point. Otherwise the crack patterns were characteristic of flexural failures and indicated no evidence of potential torsional failures.

In addition to strength the designer is frequently concerned with deflection characteristics. Fig. 8 shows the moment-deflection behavior of Beam B18-2. The shapes of these deflection curves are typical of the test results regardless of the L/b ratio. It is seen that slight lateral deflection occurred during the first increment of loading and continued to increase, without loss of stability, throughout the test. The presence of a twisting deformation is indicated by the difference between the lateral deflections at the top and at the bottom of the beam.

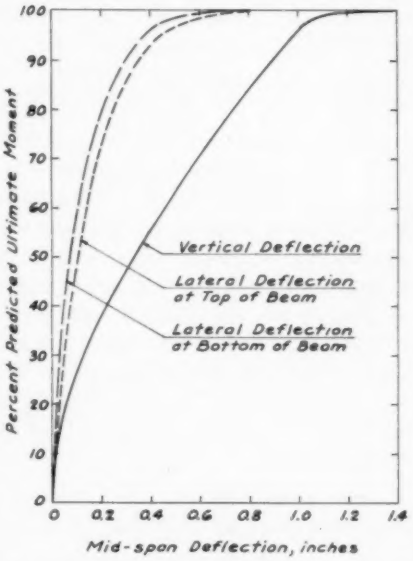

Fig. 8—Deflection behavior of Beam B18-2

This lateral deflection behavior may be ascribed to initial imperfections and to small eccentricities of the applied loads. It had no significant effect on flexural capacity and should not be interpreted as evidence of instability prior to flexural failure.

Table 2 indicates a considerable range of lateral deflections at the yield point. There appears to be little more than a qualitative correlation between lateral deflection under load, and the L/b ratio or initial lateral deflection. It may be noted that seven of the beams exhibited larger lateral deflections at the bottom than at the top and that lateral deflections exceeded vertical deflections in two tests.

INELASTIC BUCKLING ANALYSIS

The test results above constitute the main contribution of this paper. In an attempt to generalize this somewhat limited evidence the following approximate buckling analysis is presented.

Many factors affect the lateral stability of a reinforced concrete beam. One is the contribution of longitudinal steel reinforcement to the lateral and torsional rigidities of the beam. Others concern the effects of the inelastic stress-strain properties of concrete, cracking, and shear reinforcement. The effects of creep are a factor in stability studies of concrete members subjected to sustained loading. Finally, the location of loads and the conditions of end support have a considerable influence on the lateral stability of beams. It is evident that a rigorous analysis of this problem, considering all of the above factors, would be cumbersome if not impossible. It is the purpose of this section to propose conservative simplifying assumptions leading to an approximate expression for the critical buckling moment of a rectangular reinforced concrete beam subjected to uniform bending.

The buckling moment for an elastic, homogeneous beam was given in Eq. (1). This relation is analagous to the Euler formula for elastic column buckling. Obviously, Eq. (1) cannot be applied directly to a concrete beam since such a beam is neither homogeneous nor elastic. It is, therefore, necessary to establish the conversion from an elastic to an inelastic relation. This is done by replacing E and G by appropriate values in the inelastic range in analogy with the Engesser-Shanley tangent modulus theory for inelastic column buckling. This theory states that the Euler column formula predicts bifurcation of equilibrium in the inelastic range if the tangent modulus is used in place of the elastic modulus. (For all practical purposes, the load at which bifurcation of equilibrium occurs is identical with the load causing buckling.)

Correspondingly, in the inelastic range Eq. (1) for the buckling moment may be written in the form

$$M_{cr} = \frac{\overline{E}}{\sqrt{2\,(1+\mu)}} \times \frac{\pi}{L}\sqrt{I_y K_t} \qquad (2)$$

in which μ is Poisson's ratio and $\bar{E}$ is a reduced modulus related to (a) the shape of the cross section, (b) the stress-strain properties of the material, and (c) the extreme (compression) fiber strain caused by M_{cr}. The transition from Eq. (1) to Eq. (2) and the assumptions involved are outlined in the appendix, where it is shown that for a rectangular beam $\bar{E}$ equals the secant modulus, E_{sec}, corresponding to the extreme (compression) fiber strain. It is pertinent to note that M_{cr} and E_{sec} are both functions of the extreme compression fiber strain.

It remains to evaluate the geometric properties I_y and K_t in Eq. (2). Assume temporarily that the beam has an effective depth h. Then considering a rectangular section with $b < h$,* I_y and K_t may be written

$$I_y = \frac{1}{12} b^3 h \qquad (3a)$$

$$K_t = \tfrac{1}{3} b^3 h\,(3\lambda) \quad b < h \qquad (3b)$$

where λ is a function of the ratio h/b. For practical purposes one may take

$$\sqrt{3\lambda} = 1 - 0.35 \frac{b}{h} \qquad (4)$$

and so obtain the result

$$M_{cr} = \frac{\pi}{6\sqrt{2\,(1+\mu)}} \times E_{sec} \times \frac{b^3 h}{L}\left(1 - 0.35 \frac{b}{h}\right) \qquad (5)$$

One may conservatively compute the lateral and torsional rigidities of a cracked concrete beam by considering only the compression area of the concrete.

This assumption neglects any contribution of the longitudinal and shear reinforcement, and of the concrete below the neutral axis. The longitudinal reinforcement is implicitly considered only in that it affects the location of that axis. Using this assumption, I_y and K_t are given by Eq. (3a) and (3b) if h is taken as the depth c of the compression area. Then with the substitution $h = c = kd$, Eq. (5) may be written

$$M_{cr} = \frac{\pi}{6\sqrt{2\,(1+\mu)}} \times \frac{E_{sec}}{L} b^3 d \left(k - 0.35 \frac{b}{d}\right) \qquad k > \frac{b}{d}$$

and with $\mu = 0.16$ this becomes[7]

$$M_{cr} = 0.34 \frac{E_{sec}}{L} b^3 d \left(k - 0.35 \frac{b}{d}\right) \qquad k > \frac{b}{d} \qquad (6)$$

*In later developments pertaining to reinforced concrete beams it will be seen that h may be less than b (kd less than b), in which case b and h should be reversed in Eq. (3b) and (4).

It is proposed that Eq. (6) be used as an estimate for the critical buckling moment of an initially straight rectangular reinforced concrete beam subjected to uniform bending over a span L.

It should be noted that Eq. (6) is a conservative approximation in that it neglects the increase in the rigidities of a beam due to reinforcement and the uncracked concrete below the neutral axis of the beam. In regard to torsion, Cowan concludes that longitudinal reinforcement increases the torsional stiffness of reinforced concrete beams, but that "this increase is generally too small to be considered in practice" (Reference 8, p. 18). In regard to lateral bending, unless the longitudinal reinforcement is purposely placed to resist lateral bending (i.e., distributed along the sides of the beams), its contribution to the lateral rigidity of the beam is limited, particularly in slender beams which have a small width to depth ratio.

Ernst has investigated the torsional properties of reinforced concrete beams with variable quantities of shear reinforcement.[9] His experimental results indicate that for small torsional moments, the torsional behavior of a reinforced concrete beam is not significantly affected by shear reinforcement. This statement applies to torsional moments less than those causing diagonal tension cracks. One is therefore justified in neglecting reinforcing steel in the rigidity calculations leading to Eq. (6), at least as an approximation.

It is pertinent to distinguish between buckling of an "ideal" (initially straight, concentrically loaded, free from imperfections) beam and instability of a "real" (initially bowed, eccentrically loaded, imperfect) beam. Eq. (6) gives critical buckling moments for an ideal beam subjected to pure bending. A real beam becomes unstable at loads somewhat less than an identically loaded ideal beam. The application of loads above the shear center of a beam reduces the critical buckling load while loads applied below the shear center have an opposite effect. A rigorous analysis of the influence of initial bow, eccentric loading, and loads applied above the shear center on the stability of reinforced concrete beams is beyond the scope of this discussion.

Creep would appear to have a twofold effect on the stability of reinforced concrete beams subjected to sustained loads. Lateral deflections increase due to creep. In addition the secant modulus E_{sec} decreases as creep strains increase. Each of these effects would impair the stability of the beam. In considering the stability of reinforced concrete arches subjected to sustained loads, it has been suggested that creep effects may be approximated by using a reduced value of the concrete modulus.[10] It seems reasonable to use a similar procedure for concrete beams without compression reinforcement. Washa and Fluck have reported significantly reduced creep deflections for beams with compression reinforcement.[11] It is therefore reasonable to predict smaller reductions in buckling moments due to creep for compression reinforced beams than for similar beams without compression steel.

Finally, Eq. (6) applies specifically to beams subjected to a constant bending moment over the full span L and simply supported at the ends of the span. A discussion of other loading and support conditions is omitted here for brevity. However, it may be stated that the loading and support conditions on which Eq. (6) is based are more severe from the standpoint of stability than most conditions encountered in practice.[4] This statement is true for both elastic and inelastic stress-strain conditions. In fact, if one considers the inverse variations of flexural rigidity with bending moments in the inelastic range, it is evident that a beam is more stable than calculations based on the minimum rigidity at a point of maximum moment would indicate.

It may be concluded that, under most conditions, a concrete beam will remain stable if the maximum bending moment in the beam is less than the critical buckling moment given by Eq. (6). The reduction in the stability of a beam due to initial bow, eccentric loading, and loads applied above the shear center is probably cancelled approximately by the fact that Eq. (6) is conservative for most loading and support conditions met in practice.

APPLICATION OF BUCKLING ANALYSIS

For test verification and possible design application it is desirable to construct curves of laterally unsupported span length versus critical buckling moments. This section discusses the construction of such a curve for the beam section and material properties of the previously reported tests.

As indicated, the values of M_{cr} and E_{sec} in Eq. (6) correspond to the same value of the extreme compression fiber strain ϵ_c. This implies that to use Eq. (6) one must first establish the relation between M and E_{sc} throughout the entire loading range from zero to ultimate, since buckling may occur at any load in this range, depending on unsupported length. Both the area under, and the slope of, the stress-strain curve used to establish this relation must conform as closely as possible to actual stress distributions in concrete beams since this curve determines both M and E_{sec}. Thus simplified stress blocks such as a trapezoid or rectangle are not suitable for buckling calculations.

The M-E_{sec} relation may be established using a modification of Stüssi's flexural theory. Since this theory has been discussed at length by previous authors, notably Hognestad and his associates,[12,13] only the results of its application will be given here. The flexural stress-strain relation shown in Fig. 9 and the materials properties in Table 3 were used as a basis for estab-

TABLE 3—MATERIALS PROPERTIES USED IN BUCKLING CALCULATIONS

Properties of concrete
f_c' = 4290 psi
K_3 = 0.85
β = 2
γ = 0.25
ϵ_u = 3.8 mills
ϵ_0 = 1.9 mills
E_c = 3.83 × 10^6 psi
Properties of steel
f_{sy} = 43,800 psi
ϵ_{sy} = 1.53 mills
E_s = 28.8 × 10^6 psi

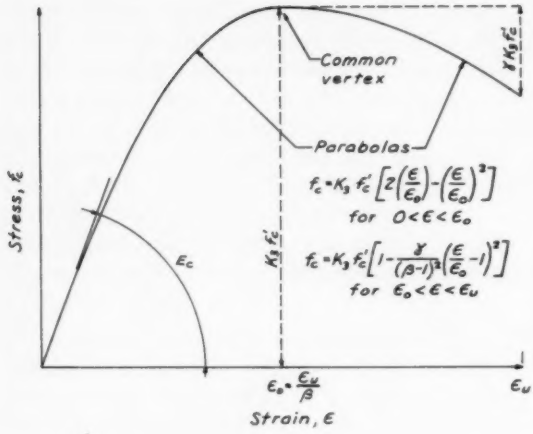

Fig. 9—Assumed stress-strain relation for concrete in flexure

lishing the M-E_{sec} relation. The assumed flexural stress-strain curve and the particular values of the parameters β and γ which determine the shape of this curve were chosen to approximate the measured flexural stress distributions reported in Reference 12.

Using Stüssi's flexural theory, data for the moment and steel strain curves in Fig. 10 were calculated. These curves assume that the steel remains elastic up to the yield point. As indicated in Fig. 10 a yield strain of 1.53 mills for the tension steel in the test beams corresponds in this analysis to a yield point bending moment of 183 kip-in. which is 8 percent on the conservative side compared with observed test beam behavior (see Table 2). Portions of the moment curve in Fig. 10 above 183 kip-in. correspond to larger steel yield strengths than those used in the testing program. A secant modulus curve based on the assumed flexural stress-strain relation is also included in Fig. 10.

Having simultaneous values of E_{sec} and M from Fig. 10 one may use Eq. (6) to estimate critical buckling moments of the test beams.

With $k = 0.382$ (a conservative and minimum value prior to tensile steel yielding), $b = 2.5$ in., $d = 11.25$ in., this equation reduces to

$$M_{cr} = 1.51 \frac{E_{sec}}{L} \quad \ldots\ldots\ldots\

from the assumed stress-strain relations as shown on Fig. 10. The maximum unbraced test span was 18 ft while M = 183 kip-in. is seen to correspond to L = 22.2 ft. Thus, Fig. 11 indicates that none of the test beams were sufficiently long to produce instability failure prior to reaching the yield point, which agrees with the test observations. Assuming the validity of the buckling curve, the minimum unbraced span length at which buckling would precede steel yielding for the test beams is 22.2 ft, corresponding to an L/b ratio of 106.

Some observations concerning the stability provisions of the current ACI Building Code (ACI 318-56) may be made from Fig. 10 and 11. The curves in these figures indicate that the limiting L/b ratio of 32 stipulated by Sec-

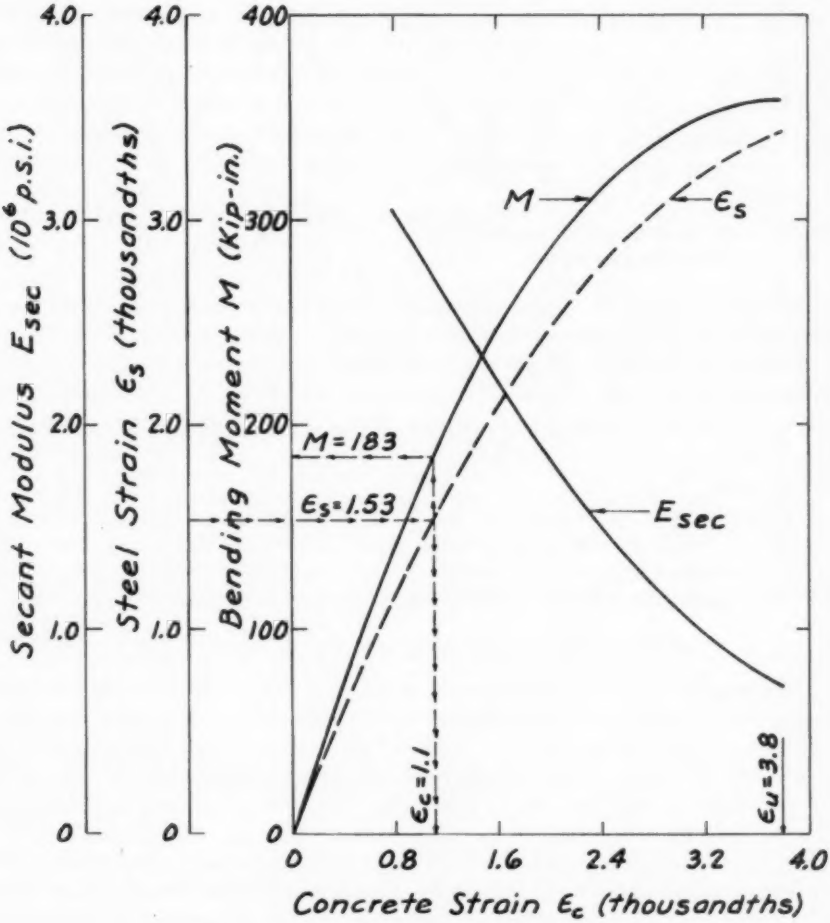

Fig. 10—Theoretical behavior of test beams

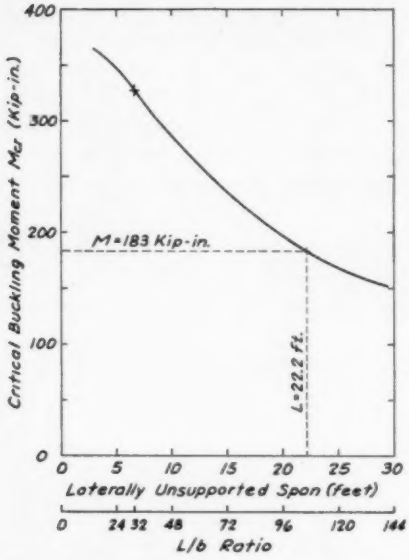

Fig. 11—Theoretical critical buckling moments for test beams

tion 704 of the Code is devoid of any significance for ordinary strengths of steel and concrete, even for beams of the unrealistically extreme cross-sectional dimensions which were used in the tests. It is seen from Fig. 11, on the other hand, that $L/b = 32$ would correspond to a critical moment of 328 kip-in. For the test beams Fig. 10 shows that this moment would be reached only if a steel strain of 2.93 mills were developed elastically, i.e., at an elastic steel stress of about 85,000 psi, which is possible only with high strength steels. This illustrates the fact that the higher concrete strains which can be developed prior to steel yielding with high strength reinforcements can lead to a significant reduction of E_{sec} and a corresponding reduction of the L/b ratio at which lateral buckling can occur.

Correspondingly, it appears justified to liberalize considerably the present limitation of $L/b = 32$ in the ACI Code when applied to beams reinforced with steel of 40,000 to 60,000 psi yield point. At the same time, since reinforcement with yield values of the order of 80,000 psi is in the process of becoming available, it would appear desirable to impose more stringent limitations on unbraced length for beams reinforced with such high strength steels.

It should be emphasized that the theoretical results of this investigation on which the above remarks are based are limited in scope and have not been experimentally verified, except to the extent that they agree with the fact that no buckling failures were obtained in any of the tests up to $L/b = 86$.

SUMMARY AND CONCLUSIONS

Ten slender beams with d/b ratios of 4.5 and L/b ratios from 28.8 to 86.4 were tested using specially designed loading rigs which eliminated any significant lateral and torsional restraint at the load points. The beams were loaded at the quarter points and were laterally unrestrained over their full span between supports. All of the beams failed in simple, vertical bending due to yielding of the tension steel, at loads in close agreement with the predictions of ultimate strength theory. There was no evidence of any reduction in strength due to laterally unsupported span length even though the largest L/b ratios were 2.7 times as large as permitted by the limitations of the current ACI Building Code (ACI 318-56).

Since the test beams were dimensionally more extreme, i.e., more conducive to lateral buckling, than beams likely to occur in practice, it is concluded that the L/b limitations of Section 704 of the ACI Building Code are excessively conservative when applied to realistically dimensioned laterally unbraced beams reinforced with ordinary strength steel. However, it was shown that more stringent L/b limitations are advisable when high strength steel is used than for ordinary reinforcement.

An approximate analytical method was suggested for estimating the critical buckling moment of a simply supported, initially straight, concentrically loaded, rectangular, reinforced concrete beam. It is believed that this method gives conservative estimates for most conditions met in practice. The proposed buckling theory was used to predict the critical buckling moments of the test beams. The theory correctly predicted that none of the test beams would buckle prior to yielding of the tension steel.

It should be noted that the testing program involved only short time loading, one beam cross section, one steel ratio and steel strength, one loading system, and one concrete strength. Further studies are desirable to verify the general applicability of the proposed buckling relation.

ACKNOWLEDGMENT

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APPENDIX—INELASTIC BUCKLING MODULI

An analytical investigation of the lateral stability of reinforced concrete beams must consider the inelastic stress-strain properties of concrete. In particular, one must determine what values of the elastic constants E and G apply in the inelastic case. It is the purpose of the appendix to consider this problem. For simplicity, lateral buckling will be considered without particular reference to reinforced concrete. It will be assumed that all members are homogeneous, initially straight, and free from imperfections, and that all loads are applied concentrically.

Consider a slender beam with constant symmetrical cross section bent about its major principal axis (X axis) by equal couples applied at the ends of the beam [Fig. A-1(a)]. The ends of the beam are supported so as to prevent rotation of the end sections about the longitudinal beam axis (Z axis). Otherwise the beam is unrestrained. The beam material is assumed to have equal stress-strain properties in tension and compression [Fig. A-1(b)]. The stress-strain relation is assumed to be inelastic and is represented in the general form $\sigma = g(\epsilon)$ where σ and ϵ denote stress and strain respectively. The tangent modulus E_t is then defined as

$$E_t = \frac{d\sigma}{d\epsilon} \qquad \text{(A1)}$$

Plane sections are assumed to remain plane during bending and the stress-strain properties of the material for axial loading are assumed to be valid for flexure. The symmetrical cross

section of the beam is assumed to have the general shape indicated in Fig. A-1(c) where $w(y)$ denotes the width of the beam at a distance y from the horizontal centroidal axis and h is the depth of the beam.

Incipient lateral buckling occurs in the inelastic stress range at a moment M_{cr} of

$$M_{cr} = \frac{\pi}{L}\sqrt{\overline{EI}_y \overline{G}K_t} \quad \text{(A2)}$$

where $\overline{E}$ and $\overline{G}$ represent reduced moduli which are related to the stress-strain curve and the strains caused by M_{cr}. The problem of inelastic buckling is reduced to evaluating $\overline{E}$ and $\overline{G}$.

In discussing the lateral stability of metal I-beams stressed beyond the proportional limit, Timoshenko[3] and Bleich[14] have proposed that the tangent modulus corresponding to the maximum extreme fiber stress be used for $\overline{E}$, i.e.

$$\overline{E} = E_t \quad \text{(A3)}$$

and that $\overline{G}$ may be approximated by

$$\overline{G} = \frac{\overline{E}}{E} G \quad \text{(A4)}$$

In a recent series of tests of aluminum I-beams, Clark and Jombock[15] found that Eq. (A2), (A3), and (A4) gave reasonably accurate estimates of observed buckling moments (for uniform bending). However, one is not justified in using Eq. (A3) for rectangular sections since their flexural strength is not derived from material concentrated in "flanges." The following discussion would establish Eq. (A3) as a special case of a more general expression for $\overline{E}$.

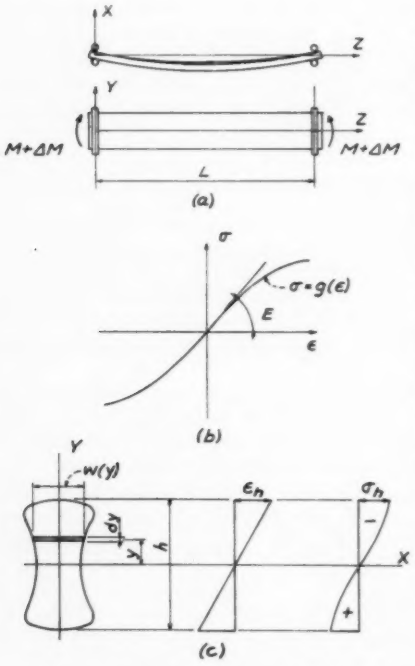

Fig. A-1—Inelastic lateral buckling

The product $\overline{EI}_y$ in Eq. (A2) is the lateral flexural rigidity of the beam at the instant of incipient buckling. It is assumed that, according to Shanley, bifuraction of equilibrium is a convenient and only slightly conservative criterion for incipient buckling. One is then led to consider the lateral flexural rigidity of the beam of Fig. A-1 at the instant when bifurcation of equilibrium occurs. Before evaluating $\overline{EI}_y$ it is pertinent to describe in detail the relations between the stresses and strains involved in bifurcation of equilibrium.

Let M represent the largest moment the beam can resist without deflecting laterally. By analogy with the tangent modulus theory, a small increase in bending moment from M to $M + \Delta M$ causes a small but stable rotation and lateral deflection of the beam. As ΔM approaches zero, the straight and deflected forms of the beam approach coincidence so that M is the moment at the instant when bifurcation of equilibrium occurs.

Again by analogy with the tangent modulus theory, the moment ΔM causes the strains

at every point in the beam to increase in absolute value.* If $\Delta \epsilon$ represents the increase in strain at any beam fiber due to the moment ΔM then the increase in stress $\Delta \sigma$ at this fiber is approximately $\Delta \sigma = E_t \Delta \epsilon$ where E_t corresponds to the strain at the fiber caused by the moment M. When bifurcation occurs the stresses $\Delta \sigma$ and strains $\Delta \epsilon$ vary across the width of the section. The resistance of the beam to lateral deflection is associated with these variations in $\Delta \sigma$ and $\Delta \epsilon$.

One is now in a position to evaluate the lateral flexural rigidity of the beam, $\bar{E}I_y$, at the instant of bifurcation of equilibrium. Consider an element of area in the beam cross section [Fig. A-1(c)] of width $w\,(y)$ and height dy at a distance y from the neutral axis of the beam. The contribution of this element to the lateral flexural rigidity of the beam may be expressed as

$$d(\bar{E}I_y) = E_o \left[\frac{1}{12} w^3(y)\right] dy$$

where E_o is the modulus relating the stresses and strains associated with lateral deflection. The preceding discussion indicates that $E_o = E_t$ so that

$$\bar{E}I_y = \frac{1}{12} \times 2 \int_0^{h/2} E_t w^3(y) dy$$

is the lateral flexural rigidity of the beam associated with bifurcation of equilibrium.

To evaluate $\bar{E}$, one may express I_y as

$$I_y = \int_A x^2 dA$$

$$= 4 \int_0^{\frac{1}{2}h} \int_0^{w(y)/2} x^2 dx dy$$

$$I_y = \frac{1}{6} \int_0^{h/2} w^3(y) dy$$

and so obtain the result

$$\bar{E} = \frac{\displaystyle\int_0^{h/2} E_t\, w^3(y) dy}{\displaystyle\int_0^{h/2} w^3(y) dy} \qquad \text{(A5)}$$

Eq. (A5) is a general expression for the reduced modulus $\bar{E}$ and is based on an analogy with the tangent modulus theory for columns. It is seen that the shape of the cross section has an important influence on $\bar{E}$. For instance, neglecting the thin web, Eq. (A5) gives $\bar{E} = E_t$ for an I-section, in agreement with Eq. (A3). For the rectangular section, one obtains from Eq. (A5)

*If the lateral deflection is small but finite, this statement is not strictly valid for beam fibers in a small band near the neutral axis. Strain reversal will occur in this band but the stress-strain curve is nearly elastic for the small strains involved. Therefore strain reversal near the neutral axis will have only a minor effect on the lateral flexural rigidity.

$$\overline{E} = \frac{b^3 \int_0^{h/2} E_t dy}{b^3 \int_0^{h/2} dy}$$

and with the substitutions

$$y = \tfrac{1}{2} h \frac{\epsilon}{\epsilon_h}$$

$$dy = \tfrac{1}{2} \frac{h}{\epsilon_h} d\epsilon$$

derived from the strain diagram in Fig. A-1(c) it follows that

$$\overline{E} = \frac{1}{\epsilon_h} \int_0^{\epsilon_h} E_t \, d\epsilon$$

Finally from Eq. (A1) one obtains

$$\overline{E} = \frac{1}{\epsilon_h} \int_0^{\epsilon_h} \frac{d\sigma}{d\epsilon} \, d\epsilon$$

$$= \frac{1}{\epsilon_h} \int_0^{\sigma_h} d\sigma$$

$$\overline{E} = \frac{\sigma_h}{\epsilon_h} = E_{sec}$$

where σ_h and ϵ_h are the extreme fiber stresses and strains respectively. This interesting result indicates that the modulus for lateral buckling of a rectangular section is the secant modulus E_{sec} corresponding to the extreme fiber strain.

No direct method of evaluating the reduced shear modulus is evident to the writers. One must therefore resort to somewhat arbitrary approximations, such as Eq. (A4). Substitution of this relation in Eq. (A2) gives

$$M_{cr} = \overline{E} \sqrt{\frac{G}{E}} \times \frac{\pi}{L} \sqrt{I_y K_t}$$

Using the known relation

$$\frac{G}{E} = \frac{1}{2(1 + \mu)}$$

where μ is Poisson's ratio, one obtains Eq. (2) as an approximate expression for the critical inelastic buckling moment of a beam subjected to uniform bending. The reduced modulus $\overline{E}$ in this equation may be evaluated for a given beam section using Eq. (A5) if the stress-

strain curve and the extreme fiber strain are known. It is equal to E_{sec} for rectangular, and to E_t for I-shaped beams. Note that according to the assumptions made here, μ corresponds to elastic values and does not vary with the extreme fiber strain.

The prime objective of the preceding study has been to show the relation between the shape of the beam section and the reduced modulus. No claim is made to a rigorous or exact solution of the problem of inelastic lateral buckling. In essence the study consists of a rational application of the tangent modulus theory to this problem.

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Title No. 56-15

Chicago's 39-Story R/C Executive House

By HENRY MILLER

This 39-story, 371 ft high apartment house is supported on drilled caissons extending into bedrock. The slender structure of high strength concrete has heavy shear walls supplemented by massive rigid frame bents in the plane of the shear walls below the sixth floor. Below grade a grid of heavy girders ties together both the rigid bents and the foundation caissons. Cantilevered balconies serve three-fourths of the apartments.

■ EXECUTIVE HOUSE—39 stories, 371 ft high—towers above the Chicago River on Wacker Drive just north of the city's Loop district. The slender steel and glass jacketed structure is Chicago's tallest reinforced concrete building, and is believed to be the tallest structure of its kind in the United States. Only four reinforced concrete buildings abroad are known to be taller; two of these are in Spain, one in Argentina, and the world record holder, 507 ft, in Sao Paulo, Brazil.

The Chicago apartment building was originally planned with a structural steel frame. However, at the time the design was completed the high price of structural steel, complicated by an anticipated year's delay in delivery, led to a decision to change the design to reinforced concrete.

BUILDING LAYOUT

The general plan dimensions (Fig. 1) are 150 x 60 ft with cantilevered balconies 6½ ft wide on both sides of the building for part of its length. This narrows a portion of the structural frame to only 47 ft, thereby accentuating already difficult wind bracing problems created by the high height-to-width ratio.

Eliminating the basement bypassed groundwater problems from the nearby Chicago River, and the first five floors are largely devoted to parking and service facilities, with shops, offices, and restaurant on the third floor at the upper level of Wacker Drive. The 38th floor houses a cocktail lounge and on the 39th floor there is an observation deck.

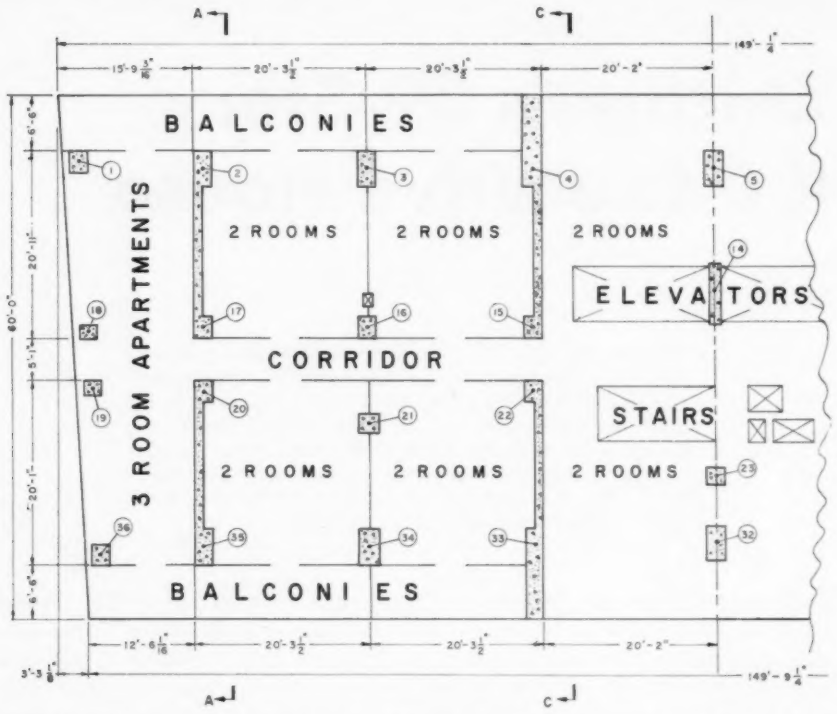


Fig. 1—Typical framing plan showing shear walls and column spacing for 7th-22nd floor. Building is practically symmetrical about center line. See Fig. 3 and 4 for sections

Thirty-two intervening floors provide 446 apartments with a total of 1216 rooms. A typical floor (Fig. 1) contains 14 individual apartments with a clear ceiling height of 8 ft. Floor slabs are of 8-in. lightweight aggregate concrete block with a 2½-in. topping and 5-in. joists spaced 21 in. on centers (except for unusual framing at lower levels). Using these lightweight block made it possible to keep all beams flat and contained within the 10½-in. slab thickness. Ceiling plaster was applied directly to the under side of the slabs. The lightweight block have desirable acoustic properties, and with the proper floor coverings they make floors and ceilings virtually soundproof.

Bathrooms and kitchens are grouped together, back to back, so that pipes and ducts including heating and cooling pipes are run in spaces between them. Each apartment is heated and cooled with individual room controls.

MATERIALS

Stainless steel bands 4 ft high comprise the building trim. At the building's center where there are no balconies, sliding windows are placed above the steel bands. Elsewhere the steel-faced curtain wall serves as balcony

HENRY MILLER (ACI member) has designed many apartment buildings, the latest being Executive House; Chicago apartment designs include 320 Oakdale Apartments, 450 Wellington St., and others. He also designed the tallest apartment building in Phoenix, Ariz., the Phoeniz Towers. In private practice since 1944, he is president of Miller Engineering Co., Chicago; he was also with Harza Engineering Co. for many years. His other projects include hydroelectric plants, industrial plants, and schools.

parapet. Sliding glass doors separate balcony from interior living area, and translucent dividers separate adjacent balconies.

The structure was designed with straight bars except for column ties and beam stirrups. Rail steel of 110,000 psi ultimate strength was used throughout. Since the Chicago building code allows only 20,000 psi working stress, it is believed that the factor of safety may be as high as five.

Two double elevator lifts were used during construction. One, equipped with a hopper for hoisting, lifted concrete to the floor under construction where it was transported by buggies to the point of deposit. Ready-mixed concrete, a total of about 16,000 cu yd, was used for the entire job. For the caissons and all structural members up to the 17th floor, 5000-psi concrete was specified, above that level 3000-psi strength was required. Laboratory tests showed the concrete to be well above these values at 28 days.

The forms for the tile and joist slabs consisted of 2 x 10-in. plank placed on the joist center line. The planks were supported by screw-jack shores. Forms for flat beams and columns were plywood with bracing as required. A total of four sets of shores were used with one set in the process of erection. One floor was formed, reinforcement set, and all piping and electrical work roughed in and concreting done in 3 working days.

CAISSON FOUNDATIONS

Test borings of the subsoil at this old river bed site indicated very bad soft clay for about 40 to 50 ft from the upper level of Wacker Drive, getting stiffer as it extended deeper. Bedrock was found about 114 to 118 feet below the ground surface. Loads were so large that bearing on any stratum other than bedrock was physically impossible. The 57 caissons drilled were therefore extended 3-8 ft down into bedrock. Each caisson, 1 meter in diameter had an effective area of 1194.6 sq in. With the 5000-psi concrete used, the carrying capacity of each caisson was figured at 1125 psi for a total of 1,350,000 lb each. To avoid excessive localized stresses the top 15 ft of all caissons were reinforced as tied round columns with the bars extending up into the underground girder system. Furthermore, since the bottom of reinforcement is some distance below the softest material, it is believed that enough passive earth pressure is developed to overcome all bending to which the caisson shaft may be subjected.

The Benoto drilling and excavating machine, invented in France and recently brought to the United States, drilled and concreted all caissons in about

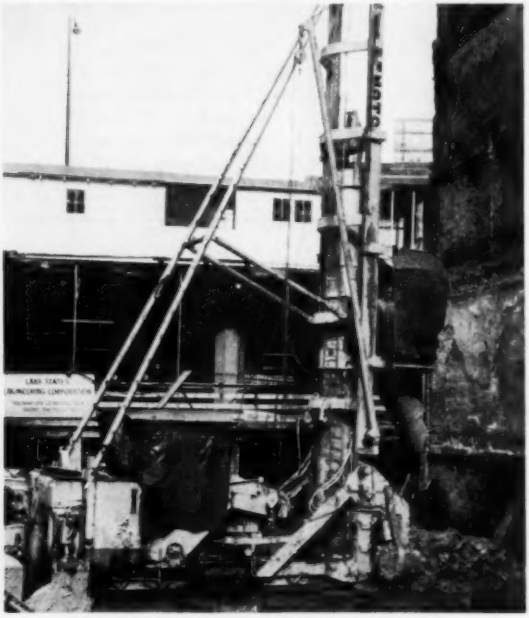


Fig. 2—The Benoto caisson drilling rig in operation

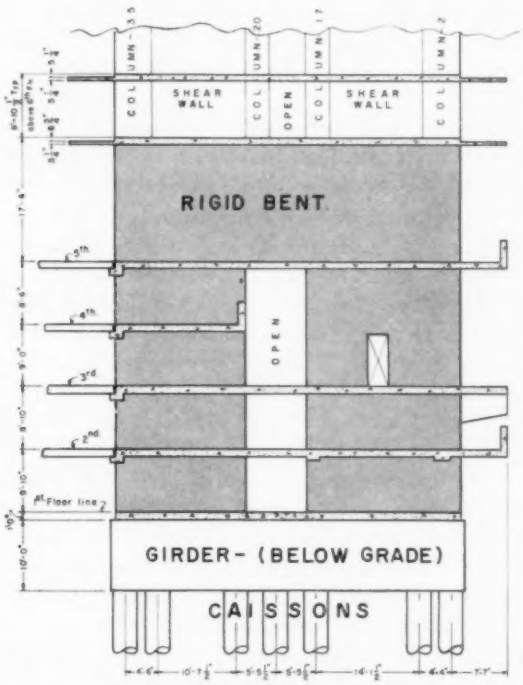
90 days. This Benoto caisson boring machine (Fig. 2) moves about the job from caisson to caisson under its own power. Using attached skids operated hydraulically, the machine puts itself in position, then with hydraulic jacks at each corner it is leveled to bring the boring operation into a vertical line. All operations are controlled with hydraulic power from a 120-hp diesel engine. The machine is designed to sink shafts about 39 in. (1 m) in diameter to a maximum depth of 350 ft. Before concrete is placed the shafts are kept dry and free of earth or debris by a continuous heavy steel casing. The casing is made up of $1\frac{5}{8}$-in. steel plate sections about 20 ft long, each weighing 1400 lb. Each length is connected to the casing below by machined rings that leave the connection smooth inside and outside. The entire length of casing is rocked back and forth with a rotating motion while it is forced down. The bottom of the lower section of the casing has a keen edge capable of cutting almost anything encountered underground. As the sinking of the casing progresses, the material within is removed. For soft material a clam shell operating within the casing is used. For rock or hard strata, a hammer-grab weighing about 3000 lb is dropped several times with its jaws open to break up the material before biting into and hoisting it to the top. Operating in this fashion one hole was drilled 115 ft to bedrock in as little as $11\frac{1}{4}$ hr.

After the shaft is cleaned of all debris and after it has been inspected and approved, the hole is filled with concrete. One load from an 11-cu yd transit mixer fills about 35 ft of caisson. During the filling operations the casing is

pulled out. Again it is rocked back and forth as well as up and down. By keeping the bottom of the casing more than 10 ft below the top surface of the concrete the action helps pack the concrete into voids of the adjacent ground. Thus cave-ins and earth intrusion into the concrete are prevented. For the present this Benoto machine is equipped to drill caissons of 1-m diameter only. In places where one large diameter caisson was required, two or more 1-m caissons were used. For example, each rigid bent is supported by seven caissons.

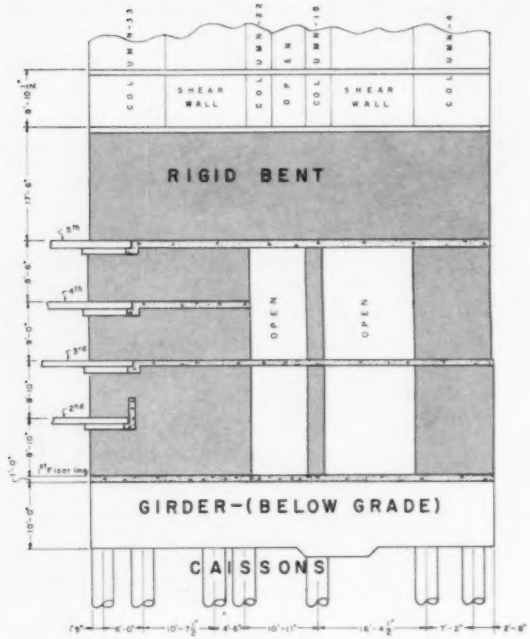
SHEAR WALLS AND RIGID BENTS

Average wind pressure for this height of building was computed to be 21.2 psf which was used for the full height. To withstand this wind load in both directions and at the same time provide proper room arrangement, every other transverse dividing partition was used as a shear wall. No shear walls were required in the long direction. The required width of the columns was found to be 26 in. The shear walls in the short direction at the narrowest part of the building consisted of two 21-ft walls divided by a 5-ft corridor in the center (see Fig. 1). The walls are 12 in. thick, reinforced in both faces, extending between two 26 in. x 5 ft columns. The two shear walls near the



SECTION A-A

Fig. 3—Section A-A. Smaller of the rigid frame bents. (See Fig. 1 for plan)



SECTION- C-C

Fig. 4—Section C-C. Rigid frame bent at the widest part of the building. (See Fig. 1 for plan)

building center are similar, but longer due to absence of balconies there. This arrangement and 5000-psi concrete in the lower portion of the building carried the combined wind and vertical loads to the sixth floor only.

Due to changes in architectural layout from the sixth floor down, the wind and vertical loads are carried by rectangular bent rigid frames (Fig. 3 and 4). The top girders of these rigid bents are 26 in. thick and extend the full story height between the fifth and sixth floors across the full width of the building. No corridor or other openings were permitted through these girders. Both the fifth and sixth floors were designed as 12-in. solid slabs to act as horizontal diaphragms and as bracing for the rigid bents. The vertical legs of the rigid bents are also 26 in. thick and are either 11 ft 6 in. or 24 ft 1½ in. wide.

The theoretical distortion or deflection of the rigid bents due to total horizontal shear at the sixth floor was figured to be about 0.3 in. This was added to the theoretical deflection of the shear walls, which were figured as cantilevers above the sixth floor, for a total theoretical deflection at the top of the building of about 4.0 in. The bottom of the rigid bent frames are tied together below the ground floor by girders which extend the full width of the building (Fig. 5). These girders are 10 ft deep and are wider than the upper part of the frames. The extra width serves two other purposes: it forms a shelf upon which the ground floor is supported; also they cover and allow the

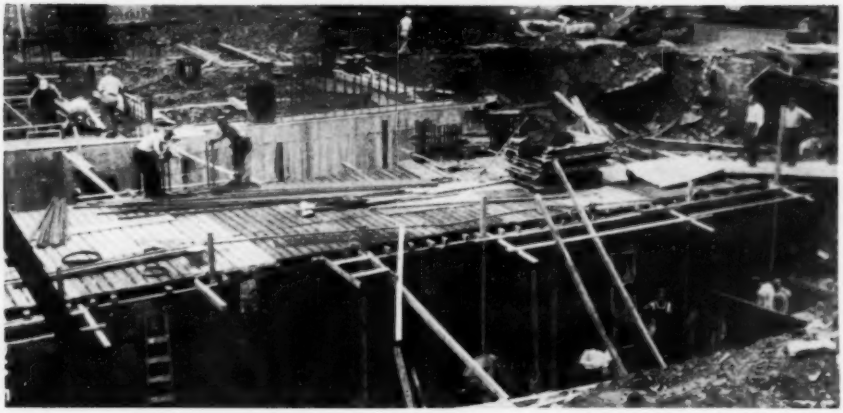


Fig. 5—Underground girders under construction. One girder is completed and forms are being set for another in the foreground. Note bars from caissons extending into the girders

caissons to extend into them. At the ends of these girders, longitudinal girders extending the length of the building are designed to tie and brace the bottom of the rigid bents and at the same time transmit the horizontal shear due to wind load into the ground.

BALCONY DETAILS

The 6 ft 6 in. cantilever balconies are 5¼ in. thick at the outer edge which is the flange width of an 8 WF 17 beam. A T-half of this beam was used along the edge of the balcony with large hairpin bars welded at the top and bottom to tie the T's to the concrete. On the inside face of the top flange, vertical 1 x 2-in. bars about 4 ft on centers are welded and extend 1 ft above the concrete surface (Fig. 6). These bars serve to anchor the balcony railings. The railing consists of stainless steel panels held in place by 2 x 3-in. aluminum hollow posts and top rail with an aluminum bottom finishing moulding. No screws, bolt heads, or nuts were to show on the finished railing. The 2 x 3-in. hollow posts with only ⅛-in. thick walls fit over the 1 x 2-in. bars provided for them with ample room for shimming into proper alignment; however, fastening the posts to the steel bars presented a problem.

None of the methods of fastening suggested were acceptable, until it was finally decided to try setting the posts in position then fill them with a premixed nonshrinking grout. Tests on the same size rods and posts made of the same materials as on the building show that after the grout set 7 days it required a 17,500-lb pull to remove the 1 x 2-in. rod from the 2 x 3-in. aluminum post. The test showed further that the rod and post acted as a unit when horizontal load was applied at the top of the post. After these tests were concluded this method of fastening the railing to the slabs was adopted. The

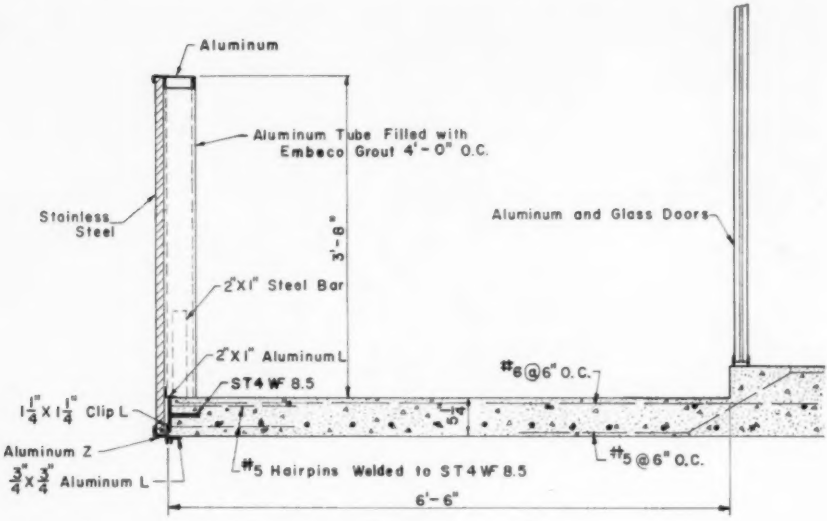


Fig. 6—Typical section through balcony

result is a neat finish between rail posts and concrete. The posts seem to be standing on the surface with no visible means of support.

ACKNOWLEDGMENTS

Architect for the building was Milton M. Schwartz and Associates, Inc., and structural design was by Miller Engineering Co., both of Chicago. C. A. Tharnstrom and Co., Skokie, Ill., was concrete contractor. Foundation work was done by Lake States Engineering Corp., Chicago.

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Title No. 56-16

Laboratory Study of Pavements Continuously Reinforced With Deformed Bars

MARTIN J. GUTZWILLER and JOSEPH L. WALING

TECHNIQUES AND RESULTS of laboratory experiments on simulated continuously reinforced concrete slabs are given. Specimens 3x28 ft, 8 in. thick, were reinforced with deformed bars, variables being bar size, spacing, and depth, and total percent of steel. The slabs, cast with preformed transverse cracks in the testing region, were tested on an elastic subgrade having a modulus of approximately 160 lb per cu in. Vertical static loads simulated traffic loads and horizontal longitudinal loads were used to simulate stresses induced by temperature changes.

Results pertaining to slab deflections, crack widths, and stresses in reinforcement are emphasized. Significant findings are compared with field observations reported in the literature, and several criteria are suggested for optimum structural design of continuously reinforced pavements.

■ RESEARCH IN PROGRESS ON CONTINUOUSLY reinforced pavements indicates the great interest in this relatively new type of pavement design. The promise of a smoother riding pavement requiring low maintenance effort for a long life is conducive to investigations leading to rational design procedures.

Field tests on continuously reinforced pavements have dealt heavily with studies of pavement deflections including crack widths[1-5] and stresses in the steel reinforcement.[1,3,4] Pavement deflections and crack widths will undoubtedly be the basis for important design criteria; and obviously the steel stresses, repetitive in nature as they are, must be maintained below the endurance limit of the reinforcement.

Pavement deflections are closely associated with subgrade "pumping." While not all subgrade materials are susceptible to pumping, it is obvious that some pavement deflection must exist to initiate pumping of the subgrades which are susceptible to such deterioration.

The purpose of continuous longitudinal reinforcement in pavements is to control the formation of cracks and to hold the cracks tightly closed. Such closed cracks prevent the ready entry of deleterious materials and provide adequate aggregate interlock for transfer of loads and forces across the cracks. After the formation of these fine cracks[6] the pavement is no longer a continuous structural unit but acts as a series of relatively rigid slab segments

connected by relatively flexible reinforced cracked sections.[7,8] Stresses in the steel reinforcement are critical at cracks.

Real knowledge of combined effects of temperature changes and wheel loads on the steel stresses, slab deflections, and crack widths for slabs reinforced with various amounts of steel in various positions and resting on various subgrades must be obtained in order to establish rational criteria for the design of continuously reinforced pavement slabs.

PURPOSE AND SCOPE OF RESEARCH

The objectives of this research were:

1. To determine by laboratory means the relationship between the percentage and position of continuous steel reinforcement and the formation, spacing, and width of cracks in reinforced concrete pavement slabs of determined thickness, caused by stresses induced by temperature, curing shrinkage, and live loads.
2. To determine the relationships of stresses and deflections to percentage and position of steel in concrete pavements continuously reinforced with deformed bars, resting on subgrades of various stiffnesses, and subjected to various combinations of live loads and simulated temperature changes.
3. To correlate the results of these laboratory experiments with reported field data wherever possible.
4. To arrive at tentative criteria for the design of continuously reinforced pavements as suggested by this study of stresses, deflections, and crack widths in pavements continuously reinforced with deformed bars.

Thus the controlled variables included in the experiments were percentage of longitudinal reinforcement, position of reinforcement, subgrade modulus, range of simulated temperature drop after casting of the concrete (longitudinal forces) and magnitude and position of simulated wheel loads. The dependent variables which were measured during the experiments were strains in the steel at and adjacent to preformed cracks, concrete surface strains and crack widths, and vertical deflections of the slabs.

EXPERIMENTAL SPECIMENS AND APPARATUS

Size of specimen

The specimens used in each phase of this research work have been of the same size. The concrete slabs were 28 ft long, 3 ft wide, and 8 in. thick. It was not intended to simulate any of the effects of lateral stresses such as warping and curling of the slab, and the 3-ft width provided enough contact surface for application of the vertical loads and permitted lateral transfer of the loads. Slab length was limited to 28 ft to permit use of facilities already available in the structural laboratory. This length was adequate for a reasonable portion of each slab (middle 10 ft) to simulate a continuously reinforced pavement and to furnish sufficient measured data. Furthermore, in this length of continuously reinforced pavement slab, a definite crack pattern could form to serve as a basis for predicting patterns which would occur in the field. The initial depth of 8 in. conforms with the actual depth used by some states in their present pavement design standards.

Materials

Concrete—The specifications for the concrete conformed to average present-day highway requirements. Due to the size of the specimens, it was appropriate to obtain ready-mixed

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Prof. JOSEPH L. WALING is in charge of structural engineering in the School of Civil Engineering, Purdue University. Following graduation from Purdue, he taught applied mechanics there, and later served as a naval architect at Norfolk Naval Shipyard, Portsmouth, Va. Returning to Purdue in 1945, he became professor of engineering sciences in 1954 and assumed his present duties in 1955. Professor Waling is an ACI member, and also is active in ASCE, Highway Research Board, and ASEE.

concrete locally. The concrete furnished was to meet the following minimum requirements:

28-day ultimate compressive strength	4000 psi
Maximum size aggregate	1½ in.
Slump	2-4 in.
Entrained air	3-6 percent

The fresh concrete was checked on delivery for slump and air entrainment. Three 6 x 12-in. cylinders and three 6 x 6 x 16-in. beams were cast for compressive and flexural strength tests.

Steel—Specifications for the reinforcement used in the laboratory specimens were as follows:

1. Bars to be hard grade deformed billet steel from the same heat and roll, and to conform to ASTM A 15-54T.
2. Deformations to conform to ASTM A 305-53T and to have the diamond pattern.
3. Bars to be chosen from commercially available sizes.

Three bar sizes were used: #4, #5, and #6. Standard tension tests were made to determine yield strength, ultimate strength, and modulus of elasticity.

Elastic subgrade

Tests were to be made on several slabs resting on elastic subgrades having the same modulus. It was anticipated that the use of a soil subgrade would introduce a variable subgrade modulus due to changing moisture content and progressive compaction during the series of tests. A subgrade was needed that would have a constant modulus during the period of the test (2 years or more), and which would be reproducible, or nearly so, at any future date.

The Firestone Industrial Products Co., Noblesville, Ind., fabricated a small test slab of rubber 41 x 66 in., 5 in. thick, for preliminary testing. This subgrade specimen was made up of ten pads 41 x 33 x 1 in. stacked brick fashion, so as to alternate the joints, and glued together. This specimen of rubber was subjected to a plate loading test and was found to have an average subgrade modulus of 175 lb per cu in. This subgrade modulus would represent a soil having medium plasticity, soft plastic clays having a modulus of approximately 50 lb per cu in. while densely graded nonplastic sandy gravels may have a modulus of 500 or more.[9]

As a result of this preliminary test on the rubber subgrade, 60 individual pads 41 x 33 x 1 in. were purchased with the request that the rubber be made slightly more flexible than that of the trial rubber slab, in order to be more nearly in the middle range of soils of medium plasticity. The pads were stacked on the laboratory floor, brick fashion, unglued, in five layers and the resulting subgrade modulus was 155 lb per cu in. Leaving the pads unglued provided greater flexibility for laying out of future subgrades of different dimensions, patterns, and moduli. For example, such a laid-up subgrade 3 in. thick has a modulus of 440 lb per cu in.

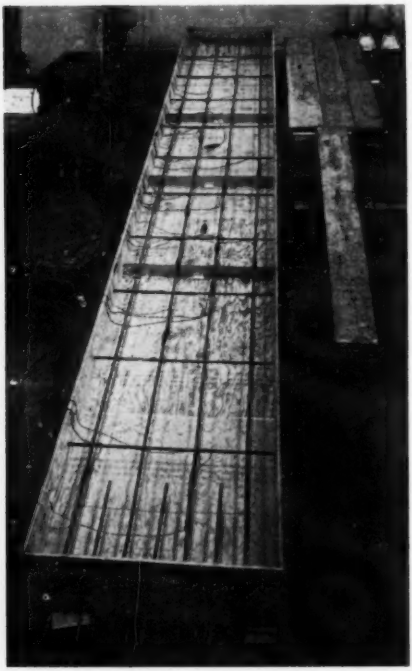

Fig. 1—Full length view of form. Note transverse metal strips used to preform cracks

Preformed cracks

A survey of literature pertaining to continuously reinforced pavements showed that the spacing of cracks developed in service varied from approximately 2 ft to 15 ft depending on the amount of reinforcement, thickness of slab, and the distance from a formed joint at the end of a test strip. A crack spacing of 5 ft was thought to be fairly representative of the average condition in the central portion of a continuously reinforced pavement; hence, three transverse cracks were preformed—one at the center of the slab and one 5 ft from the center on each side. The primary purpose of preforming these cracks was to insure that strain gages could be installed on the steel reinforcement at cracked sections.

The cracks were formed using 18-gage sheet metal strips 3 in. wide and 4 ft 6 in. long placed through vertical slots in the side boards of the form to span the form just under the longitudinal bars (Fig. 1). Two small pieces of plywood nailed to the bottom of the form on each side of the metal kept it from deflecting sideways to any great extent when the concrete was placed against it.

Form for test slab

The form consisted of three plywood carts each 8 ft long and one cart 4 ft long resting on truck casters. The four carts assembled end-to-end (Fig. 1) with side and end boards in place provided a clear space 3 x 28 ft, 8 in. deep. Holes were drilled in the end and side boards of the forms to receive the longitudinal and transverse bars. Slots were cut in the sides of the form to hold the sheet metal used to preform the partial cracks in the slabs.

Preparation for casting

Longitudinal reinforcement consisted of either #4, #5, or #6 bars as the longitudinal reinforcement, with #4 bars as transverse reinforcement. Extra deformed bar dowels were installed at the ends of the slab to provide adequate strength in fittings for applying longitudinal forces.

The 12 transverse bars placed in the slab at 2½ ft on centers served four purposes:

1. By means of threads and nuts on the ends of the bars, they tied the side boards of the form together, thus restraining the sides against spreading.
2. They supported the longitudinal reinforcement in its correct position.
3. They served as lugs for the hangers used in lifting the slab from the form and lowering it onto the rubber.
4. Inclusion of transverse steel conforms to standard practice in reinforced concrete pavements.

The strain gage stations on the bars were prepared by machining the bars down to approximately uniform diameter in a length of about 2 in. at each station. Across each of the pre-

formed partial cracks, each bar was turned down to the same diameter so as to permit a check on the load carried by the various bars at those cracks. Gages were placed on the longitudinal bars at the preformed cracks and at several other locations between the cracks. Two Type A-7, SR-4 electric strain gages were applied diametrically opposite one another, wired, and waterproofed at each of the predetermined gage stations. The longitudinal bars were placed in position with the strain gages on the sides, and were tied to the transverse steel with wire. Two temperature compensation gages mounted on short pieces of reinforcement were placed in the form. The strain gage lead wires were carefully laid out to emerge from the form at the top edge near the middle of the test specimen.

The three pieces of oiled 18-gage sheet metal were placed through the sides of the forms at the location of the preformed cracks.

Placing and curing the concrete

The ready-mixed concrete was vibrated carefully to insure filling all spaces around reinforcement, strain gages, and lead wires. After striking off and before initial set had taken place, plugs for Whittemore strain gage readings were embedded in the top surface of the slab.

Approximately 6 hr after placing, two layers of wet burlap were laid on the slab and were covered with heavy canvas. The burlap was kept continuously wet for 3 weeks, after which the cover, burlap bags, and side and end boards of the form were removed. The test cylinders and beams were cured in the same manner and were tested 28 days after casting.

Positioning slabs for test

A system (Fig. 2) was devised whereby a test slab could be carefully transferred from the form to the rubber base. Eyebolt hangers were hooked onto the projecting ends of the transverse bars in the test slab. The threaded upper ends of the hangers on each side of the slab were engaged by nuts and washers to a longitudinal timber beam which was supported parallel to the side of the slab by a steel framework over the slab. By tightening all of the nuts at the upper ends of the hangers, the slab was raised free of the form and suspended while the form was removed and the rubber subgrade was laid below. The slab was then lowered by loosening all of the nuts evenly.

EXPERIMENTAL PROCEDURES

Instrumentation

Measurement of vertical deflection—On a line 5 in. from one edge of the slab, nine 0.001-in. Federal dial indicators were suspended from a framework with the stems of each engaging a piece of sheet tin bonded to the surface of the concrete. These indicators were spaced at 20 in. on centers in the test area, with an indicator at each of the preformed cracks. The dials indicated vertical deflections of the surface of the slab at these points, when the slab was subjected to the various combinations of loading.

Measurement of surface strains and crack widths—In the test area, plugs were embedded in the surface of the concrete at 10-in. intervals on a line 8 in. from one edge of the slab. Gage holes were drilled in these plugs and a 10-in. Whittemore strain gage was used to make surface strain and crack width measurements.

Stresses in reinforcing bars—Electric SR-4 strain gages were used to determine the steel stresses. Since stresses in the longitudinal steel could be assumed to be primarily uniaxial, the strain readings obtained, when multiplied by the modulus of elasticity of the steel in the bars, gave the stresses in the bars. Type A-7 paper-backed constantan wire gages having a resistance of 120 ohms and a gage factor of 1.9 ± were used.

Two gages at each station were placed longitudinally on opposite sides of the bar, with the center of their gage filaments at the same cross section but with their leads facing in opposite directions. The two gages were used to observe any bending or eccentric load effect.

Strain readings were taken with a Baldwin SR-4 Type L strain indicator.

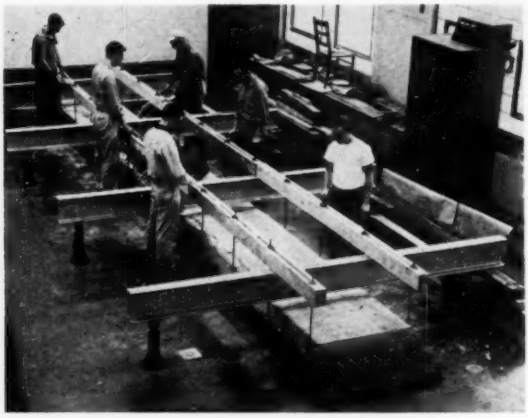

Fig. 2—Lowering of slab onto rubber base

Vertical loading

Framing for application of vertical loads (Fig. 3) to the slab consisted of a longitudinal beam supported above the longitudinal center line of the slab by four transverse bents spaced 6 ft on centers. Each bent was made up of a transverse beam clamped at its ends to vertical pipe posts which were threaded into floor inserts on each side of the slab. The longitudinal beam transmitted the reaction of the vertical load to the bents which in turn transmitted the vertical uplift to the floor inserts.

Magnitude of the vertical load, applied by hydraulic jack, was determined with a proving ring placed between the jack and the longitudinal beam. Three loading magnitudes of 5000, 10,000, and 15,000 lb were applied. The largest of these compares closely to a wheel loading for the H20-S16-44 loading which is the present standard for design of bridges and larger than the wheel load for highway pavements. It was estimated that a 15,000-lb load on the 3-ft width of test slab would produce maximum bending moments of about the same magnitude as the maximum moments produced by a pair of 16,000-lb wheel loads on a 12-ft pavement slab. The 5000-lb load was transmitted to the slab through a stack of circular steel plates of increasing size, of which the largest was 12 in. in diameter, and a piece of rubber of the same diameter located on the surface of the slab at the test position. The 10,000 and 15,000-lb loads were applied through a stack of plates, of which the largest was 18 in. in diameter, and a piece of rubber of the same diameter. The rubber served to seat the plates

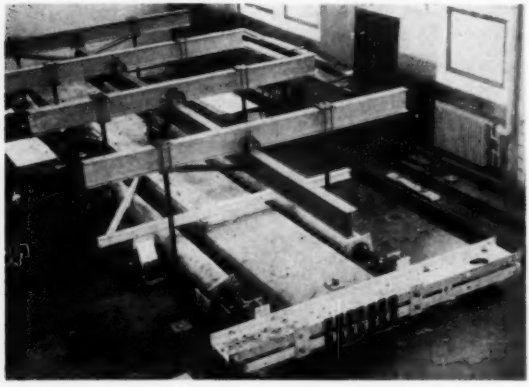

Fig. 3—Framework for vertical loading. Horizontal (longitudinal) loading equipment also in position

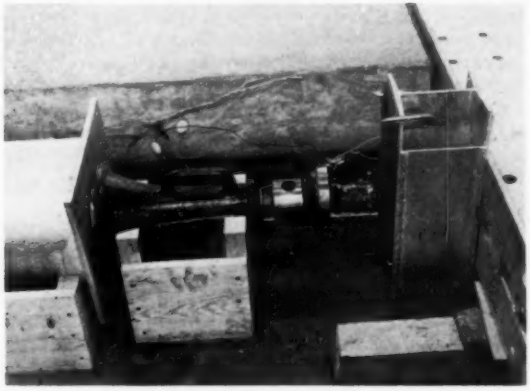

Fig. 4—Longitudinal load jack and cell

on the slab surface, thus obtaining a more uniform application of load to more nearly simulate an actual tire load. These sizes conform sufficiently closely to the equivalent contact areas of 12-in., 16-in., and 19-in. diameters respectively for 5000, 10,000, and 15,000-lb dual wheel loads as given by the Portland Cement Association.[9]

Longitudinal loading

To simulate temperature stresses, tensile stresses were induced in the reinforced slab by applying longitudinal loads to the projecting bars at the ends of the slab. To preclude the possibility that the bars might yield at the end connections rather than at a crack in the slab, end dowels of the same size as the longitudinal reinforcement were placed between the longitudinal bars. Rather than to turn down and thread the ends of the projecting bars, which would reduce their area considerably, threaded rod inserts were welded to the bar ends. These inserts were large enough so that holes up to 1 in. in diameter could be drilled in the ends and still provide enough area to develop the full strength of a #7 bar. Thus these rod inserts were reused, starting with the smallest size of bar reinforcement and drilling larger holes as the bar size was increased.

A set of two beams was placed transverse to the slab at each end with the projecting bars passing between them. Plates and nuts were fitted on the threaded ends of the rod inserts with the plates flush against the beams. Thus when the beams were forced away from the slab in the longitudinal direction, a tensile load was applied to the bars which in turn transferred a portion of it to the concrete of the slab. The nuts were adjusted so that the longitudinal load was distributed as evenly as possible among the longitudinal bars. This distribution was facilitated by SR-4 strain gages mounted on the projecting bars between the inserts and the slab end. Details of this arrangement can be seen in Fig. 3.

The longitudinal load was applied to the transverse beams by either screw jacks or hydraulic jacks placed on each side of the slab near each end. The jacks on each side of the slab worked against each other through pipe columns (also seen in Fig. 3). The applied load was measured by means of cylindrical load cells which were placed between the jacks and the transverse beams (Fig. 4). Small bearing beams placed between the load cells and the transverse beams served to distribute the load on the flanges of the transverse beams.

For the large longitudinal loads, four solid cylindrical steel load cells 4 in. long, each having a cross-sectional area of 3.14 sq in., were placed two at each end. For the small and medium loads, to obtain greater sensitivity two of the solid load cells were replaced by hollow circular stainless steel load cells 4 in. long having a cross-sectional area of 1 sq in.

Each load cell was constructed by placing four SR-4 strain gages on longitudinal elements at the quarter points of the circular cylinder. Two diametrically opposite gages were connected in series; the two pairs were then connected in parallel. The series connections elimi-

nated the effect of any bending of the cylinder and the parallel connection of the pairs in series resulted in a resistance change of the connected four gages equal to that of a single gage on a perfectly axially loaded cell. The load cells were calibrated in a 120,000-lb capacity Baldwin testing machine.

Many additional details concerning the design of the experiments and design and operation of the test apparatus are given in reports by Ficks[10] and Patterson[11].

SLAB TESTS AND RESULTS

Characteristics of the slabs tested are summarized in Table 1. The subgrade modulus was maintained constant at 155-160 lb per cu in. Vertical loads were applied at eight locations on the longitudinal center line of each slab as shown in Fig. 5.

The loading and data collection procedure was as follows:

1. A set of strain and deflection readings was taken with no load on the slab. Vertical loads of 5000, 10,000, and 15,000 lb were then applied at Position 1, and a set of readings was taken for each load. This procedure was repeated for the remaining seven vertical load positions, successively.
2. A set of strain and deflection readings was taken with no load on the slab. An initial longitudinal load of 10,000 lb was applied and a set of readings was taken. While holding this longitudinal load constant, the vertical loadings and data collection were repeated at Positions 1-8, successively, and the loads were then removed.
3. The horizontal load was increased and Step 2 was repeated for each increment. In general, increments of 10,000 lb were used.

During the entire loading sequence, measurements of concrete strain and crack widths were made by means of the steel plugs embedded at the upper surface of each slab. Gage holes were drilled in these plugs and a 10-in. Whittemore strain gage was used to make surface strain and crack width measurements. For an uncracked gage length the average unit strain was taken as the change in gage length divided by ten. When a transverse crack occurred between two plugs, it was assumed that the total change in that gage length was due to a change in the crack width. This continuous set of gage lengths thus made it possible to determine the over-all change in length of the test region due to various combinations of horizontal and vertical loads.

The longitudinal tensile loads simulated the effect of temperature drops. Correlation of the magnitude of longitudinal load to temperature drop was determined by calculations. The over-all increase in length of the central test region of a slab due to longitudinal load, as determined by the surface strain readings, was equated to the decrease in length of the same span of

TABLE 1—TEST SPECIMEN CHARACTERISTICS

Slab No.	Deformed bar reinforcement	Percent reinforcement	Depth of reinforcement below top of slab, in.	Reinforcement yield point stress, psi	Reinforcement ultimate strength, psi	Concrete compressive strength, psi
1	#4 at 9 in. on centers	0.278	4	57,300	94,600	4800
2	#5 at 9 in. on centers	0.430	4	56,300	97,500	5200
3	#5 at 7 in. on centers	0.533	4	56,300	97,500	4270
4	#6 at 7 in. on centers	0.768	4	55,800	97,600	4730
5	#5 at 7 in. on centers	0.533	6	69,500	104,400	5960
6	#5 at 7 in. on centers	0.533	5	69,500	104,400	3500

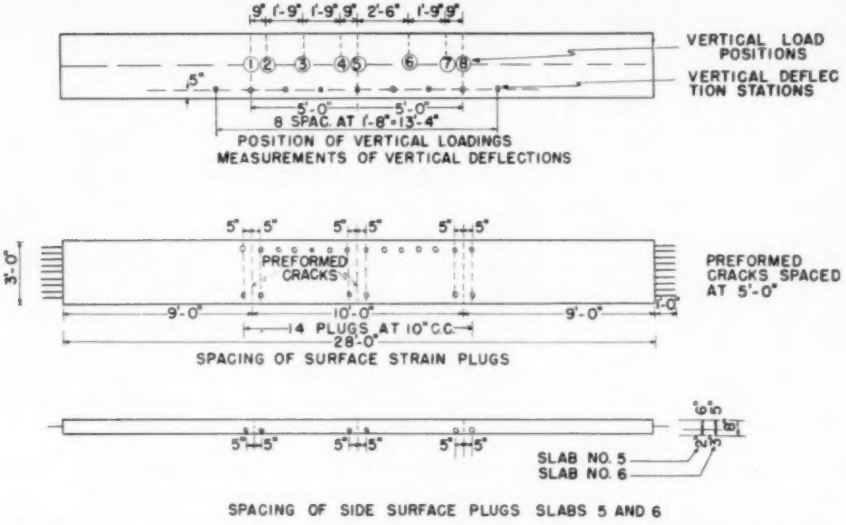

Fig. 5—Vertical loading positions and points at which strain and deflection readings were made

reinforced concrete due to a temperature drop Δt. Thus the condition of effective full restraint against longitudinal movement in a continuously reinforced slab away from the end regions was assumed. In equation form

$$e = \alpha L \Delta t \quad \text{or} \quad \Delta t = \frac{e}{\alpha L}$$

where e = elongation (in.) of central test region; L = length (in.) of central test region; α = coefficient of linear contraction—assumed to be 0.000006 in. per in. per deg F for both steel and concrete; and Δt = temperature drop (deg. F).

Details concerning the calculations necessary for the correlation of longitudinal forces and temperature decreases are given elsewhere.[6]

Vertical deflections

Vertical deflections of each slab were measured at nine stations as shown in Fig. 5. To show the effects of vertical loads on slab deflections, each slab was assumed to be straight when subjected to the various longitudinal loads with no vertical loads acting. The additional effects of 5-, 10- and 15-kip loads placed at each of the three preformed cracks were then determined. A series of graphs, similar to Fig. 6, was obtained for each slab subjected to a full range of the longitudinal loads which simulated temperature decrease. Note that some negative deflection (upheaval) was experienced at cracks a distance from applied vertical loads.

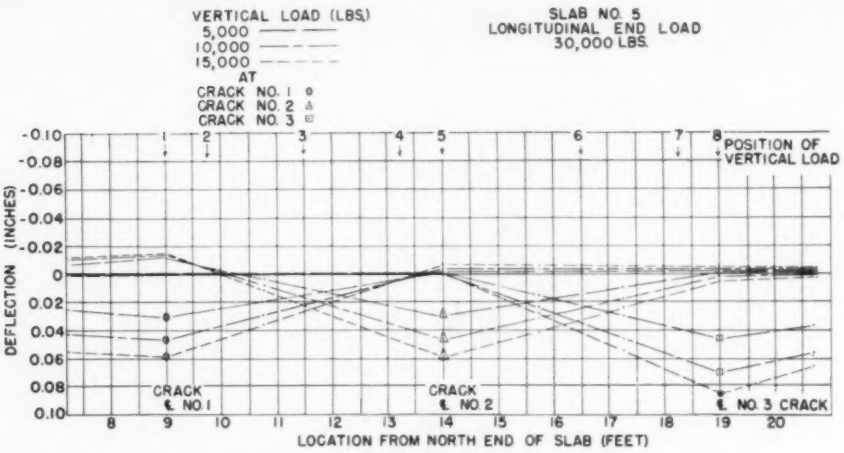

Fig. 6—Vertical deflection of Slab 5 due to combined loading at a crack

The influence of temperature drops on slab deflections can be seen in Fig. 7. Data for this figure were obtained by averaging the maximum deflections at the three preformed cracks due to 15-kip vertical loads for each longitudinal load (temperature drop) on each slab. In general it can be seen that a temperature drop of about 20 F after casting a slab tended to reduce the slab deflections due to vertical loads, while temperature decreases of 30 to 100 F tended to increase the deflections. This increase in deflections can be attributed to the loss of stiffness of cracked sections as the cracks are opened wider by decreasing temperatures. Actually, the 30 to 100 F range of temperature drops is in all probability the practicable range of slab temperature variations. Depending upon locality, a temperature decrease of 100 F or less would cause stiffening of the subgrade through freezing; then the reported data would no longer be applicable for greater temperature drops.

Fig. 7 also indicates that the percentage of reinforcement has some influence on the deflections due to 15-kip wheel loads. The slopes of the maximum deflection-temperature drop curves are affected by the number of additional cracks which develop, which in turn is governed by the percentage of steel. Slabs 1 and 2 which retained essentially the preformed crack pattern showed a steady increase in maximum deflection with increasing temperature drops beyond about 30 deg, with Slab 2 (0.430 percent steel) showing less deflection than Slab 1 (0.278 percent steel). Slabs 3 and 4 which had sufficient reinforcement to form additional cracks showed a leveling off of the maximum deflection-temperature drop curves. Here also Slab 4 (0.768 percent steel) experienced less deflection than Slab 3 (0.533 percent steel).

Fig. 8 shows the influence of temperature drop and position of reinforcement on deflections due to wheel loads. This set of curves was obtained in exactly the same manner as Fig. 7 and shows clearly the effect of lowering

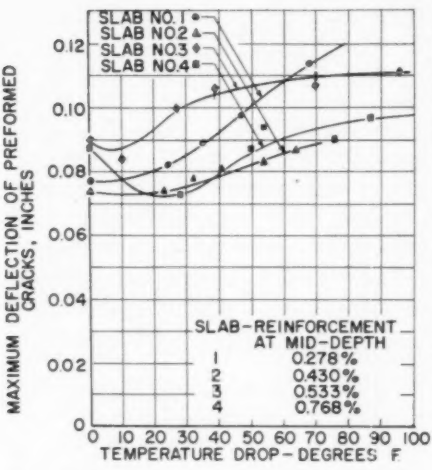

Fig. 7—Influence of temperature drop and percent of reinforcement on deflections due to 15-kip wheel loads

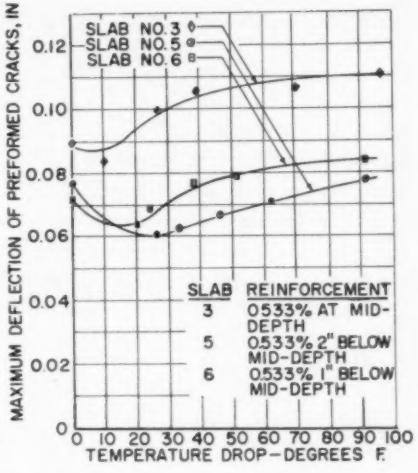

Fig. 8—Influence of temperature drop and position of reinforcement on deflections due to 15-kip wheel loads

the steel below middepth. The lower depth of reinforcement represented by Slab 5 is indeed advantageous if one considers only deflection as a criterion. Slab 6, which had its reinforcement placed 1 in. below middepth, showed a decided advantage over Slab 3 with reinforcement at middepth.

Comparison of Fig. 7 and 8 shows that Slab 6 with 0.533 percent reinforcement 1 in. below middepth offered slightly more restraint to deflection than Slab 4 with 0.768 percent reinforcement at middepth.

Perhaps more important than maximum deflection as an indication of the possibility of initiating pavement pumping is the total vertical movement (maximum deflection plus maximum upheaval) of the slab at preformed cracks. Fig. 9 shows the influence of temperature drop and percentage reinforcement on total vertical movements due to 15-kip wheel loads. In the range of temperature drops above about 60 F, Slabs 1, 2, and 3 offered about the same restraint to vertical movements while Slab 4 experienced somewhat less total movement under the 15-kip wheel loads. Fig. 10 illustrates the advantage of lowering the reinforcement steel in the slab to reduce total vertical movements due to wheel loads. A definite trend in favor of lowering the steel is indicated if one uses only the criterion of least vertical movement as an indication of an advantage. At 70 F temperature drop, Slab 5 with reinforcement 2 in. below middepth showed a total vertical movement of 29 percent less than Slab 3 with the same reinforcement at middepth; the vertical movement of Slab 6 with the same reinforcement 1 in. below middepth showed a total vertical movement 14 percent less than that of Slab 3. A comparison of Fig. 9 and 10 shows that Slab 4 (0.768 percent reinforcement at middepth)

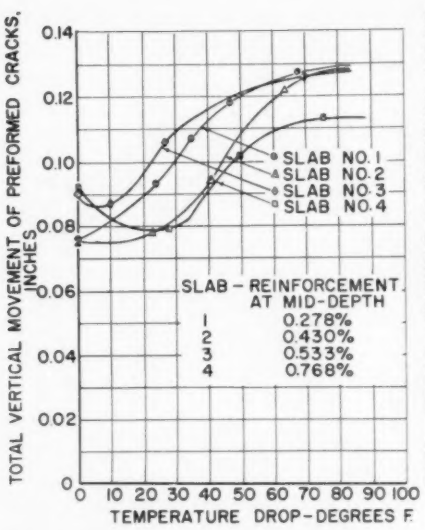

Fig. 9—Influence of temperature drop and percent of reinforcement on total vertical movement due to 15-kip wheel loads

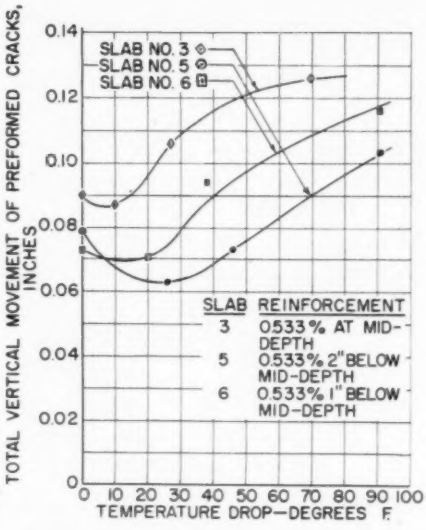

Fig. 10—Influence of temperature drop and position of reinforcement on total vertical movement due to 15-kip wheel loads

was subjected to about the same total vertical movement due to 15-kip wheel loads as Slab 6 (0.533 percent reinforcement at 1 in. below middepth).

Crack widths

It is generally agreed that for optimum design of continuously reinforced pavements, surface crack widths both at the top and at the bottom of the slab should be minimized. The importance of minimizing crack widths was explained by the late W. R. Woolley[12] while discussing "blowups" as follows:

> "The cause of blowups is not entirely certain. The compressive stress introduced by raising the temperature 100 deg in concrete having fixed ends does not exceed 2500 psi, which is hardly enough to cause blowups in standard pavement concrete. Mr. Griffin of New Jersey has given an explanation that undoubtedly applies in at least some cases. Dirt getting into a crack at the top and perhaps also at the bottom, causes the concrete to spall when the temperature rises and the cracks tends to close. After a while, as a result of spalling, the compression across the crack must be carried by a much smaller section of concrete than the original full depth. Blowups then occur due to the entire compressive force being concentrated on an area considerably less than the full cross section of the pavement.
>
> "It is recognized that dirt getting into cracks fills up space that would otherwise be available to relieve compressive forces. In a properly designed continuously reinforced pavement the cracks are held so tight that dirt does not get into them and spalling does not occur. Very likely these features will prevent many blowups that would otherwise occur."

Crack width data taken from the upper surface strain plugs with the slabs subjected to longitudinal loads (temperature drops) only, led to the study of the influence of temperature drop and percentage reinforcement on the upper surface crack widths, summarized in Fig. 11. Averages of the upper surface widths of the three preformed cracks of the first four slabs were plotted. All four slabs showed crack widths increasing somewhat linearly with temperature drops up to about 50 F. The curve for Slab 4 was influenced in this lower range of temperature variation by the additional crack which formed by curing shrinkage alone. The formation of additional cracks at temperature drops between 45 and 90 F in Slabs 1, 3, and 4 caused the curves for these slabs to flatten out, with maximum surface widths of about 0.024, 0.019, and 0.024 in., respectively, at 90 F temperature drop. The average crack width of Slab 2 which had formed no additional cracks was about 0.030 in. and still increasing at the 90 F temperature decrease. It is to be noted that the compressive (and therefore tensile) strength of the concrete of Slab 3, which experienced the most cracking, was the least among these four slabs. It is also to be remembered that the cracks formed not only as a result of temperature changes but also due to wheel loads. Thus the curves show directly the influence of the percentage of reinforcement as well as the concrete strength, and show indirectly the effects of wheel loads.

Fig. 12, obtained in exactly the same manner as Fig. 11, shows the influence of temperature drop and position of reinforcement on the upper surface crack widths. Slab 5 differed markedly from Slab 3 in that the simulated temperature drops and the vertical loads both caused many cracks to start in the lower portion of Slab 5 but caused only a few to break completely

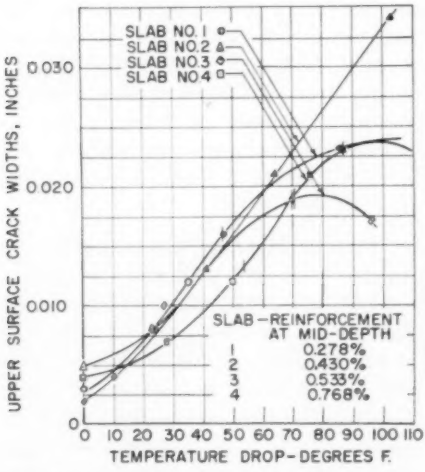

Fig. 11—Influence of temperature drop and percent of reinforcement on upper surface crack widths

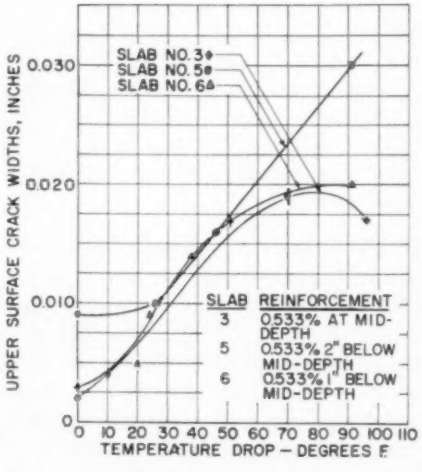

Fig. 12—Influence of temperature drop and position of reinforcement on upper surface crack widths

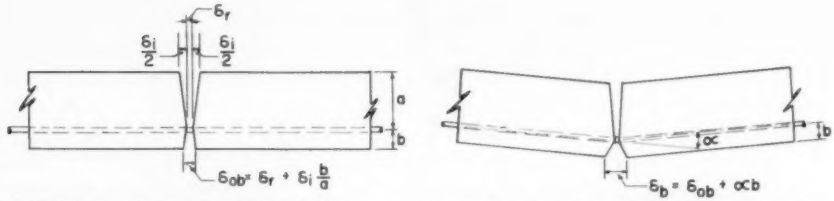

Fig. 13—Deformations at a crack

through. In fact, up to 132 F equivalent temperature drop no complete cracks in addition to the three preformed cracks at 5-ft spacing had formed. Thus the upper surface cracks of Slab 5 showed continually increasing widths throughout the practical range of temperature drops. Slab 6, with 0.533 percent reinforcement 1 in. below middepth, contained somewhat less of the partial cracks that characterized Slab 5 and had cracked completely through at one section between two of the three preformed crack planes. Thus the upper surface crack width curve for Slab 6 was quite similar to that of Slab 3. At 90 F temperature drop, the average upper surface crack widths of Slabs 3, 6, and 5 were 0.019, 0.020, and 0.030 in., respectively. This indicates that for adequate upper surface crack width control the 0.533 percent reinforcement of an 8-in. slab on a subgrade having a modulus of 160 lb per cu in. should not be placed as low as 2 in. below middepth.

In the design of a continuously reinforced pavement, the upper surface crack widths due to temperature changes alone are less important than top and bottom surface maximum crack widths due to temperature changes and wheel loads. As the wheel loads move along the pavement the slab deflects and the top and bottom crack widths vary over significant ranges. Water and dirt can work into cracks at the upper surface under the action of gravity and at the bottom surface under the pressure developed between the slab and the subgrade. Thus it is important to minimize active crack widths at both top and bottom surfaces of the slab.

The laboratory experiments involved measurement of the variation of upper surface crack widths with temperature changes and wheel loads for those slabs reinforced by steel at middepth. In addition, crack widths on the sides of the slabs at the elevation of the steel were measured on the slabs reinforced with steel below middepth. Good estimates of bottom surface crack widths were obtained in the following manner. For slabs reinforced with steel below middepth, top surface and side crack width data for temperature changes alone indicated cracks increasing in width with the distance from the steel as shown exaggerated in Fig. 13a. The measured crack width at the elevation of reinforcement is δ_r. The upper surface crack width $(\delta_r + \delta_i)$ was also measured. It was assumed that the increase in crack

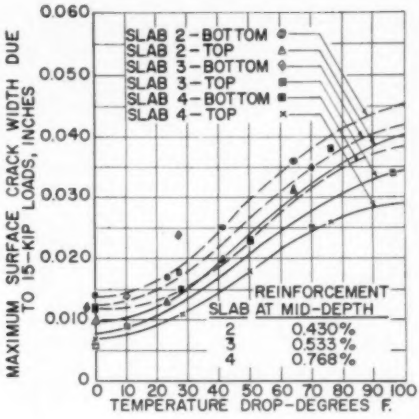

Fig. 14—Influence of temperature drop and percent of reinforcement on maximum surface crack widths due to 15-kip loads

width both above and below the reinforcement was linear. Thus the crack width δ_{ob} at the bottom surface due to temperature alone was

$$\delta_{ob} = \delta_r + \delta_i \frac{b}{a}$$

Applications of vertical loads changed the upper and lower crack widths appreciably. From the deflection diagrams (similar to Fig. 6), angle changes due to 15-kip wheel loads were scaled accurately for each of the three preformed cracks for each horizontal load on each slab. These angle changes were assumed to be centered at the reinforcement in the cracked section. When multiplied by the distance of the reinforcement from the lower surface, they gave the additional bottom surface crack widths δ_α due to the angle changes induced by the wheel loading. Thus the total bottom surface crack width δ_b due to temperature change and wheel loading is (Fig. 13b).

$$\delta_b = \delta_{ob} + \delta_\alpha = \delta_r + \delta_i \frac{b}{a} + \alpha b$$

For the special case of middepth reinforcement, the ratio b/a is equal to unity and the first two terms of the bottom surface crack width (the last member of the equality) are the same as the top surface crack width due to temperature drops alone. This is to say, for middepth reinforcement it is assumed that due to temperature changes alone, top surface crack widths are equal to bottom surface crack widths.

Fig. 14 shows the influence of temperature drop and percentage reinforcement on maximum top and bottom surface crack widths due to 15-kip wheel loads for slabs reinforced at middepth. It is seen by comparison of Fig. 14 and 11 that wheel loads do have appreciable effect on maximum top surface crack widths. A definite trend is shown toward an inverse variation be-

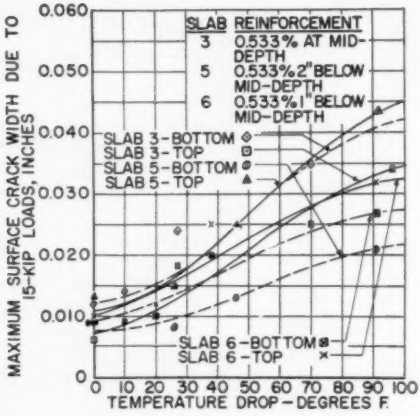

Fig. 15—Influence of temperature drop and position of reinforcement on maximum surface crack widths due to 15-kip wheel loads

tween surface crack widths (top and bottom) and percentage reinforcement, for 15-kip vertical loads. Furthermore, for middepth reinforcement the bottom surface crack widths are always greater than those at the top surface; e.g., about 20 percent greater at 70 F temperature drop. Thus middepth reinforcement does not equalize the maximum top and bottom surface crack widths.

The influences of temperature drop and position of reinforcement on maximum surface crack widths due to 15-kip wheel loads are shown on Fig. 15. The three slabs all with 0.533 percent reinforcement placed at different positions show results of striking importance. Slab 3 with steel at middepth showed top and bottom maximum crack widths as seen in Fig. 14, with the bottoms of the cracks wider than the tops. Slab 5 with its steel 2 in. below middepth showed excellent control of crack widths at the bottom surface but the poorest control at the upper surface; i.e., in Slab 5 the top surface crack widths were on the order of twice as great as the bottom surface crack widths. Slab 6 with reinforcement 1 in. below middepth had the best balance of top and bottom crack widths, with its bottom surface cracks a little narrower than its top surface cracks. Thus the set of curves in Fig. 15 shows that from the standpoint of controlling both top and bottom crack widths in an 8-in. slab reinforced with 0.533 percent steel and resting on a subgrade having a modulus of 160 lb per cu in., the best even-inch position of the reinforcement is 1 in. below middepth. Furthermore, a comparison of Fig. 14 and 15 shows that 0.533 percent steel 1 in. below middepth (Slab 6, Fig. 15) is somewhat better in controlling top and bottom surface crack widths than 0.768 percent steel at middepth (Slab 4, Fig. 14).

Steel stresses

The longitudinal steel of continuously reinforced concrete pavements is subjected primarily to uniaxial tensile or compressive stress. While the longitudinal steel does assist in the load transfer across transverse cracks in

the concrete, its main contribution is made by holding the cracks tightly closed so that aggregate interlock resists the greater portion of the shear forces at a crack.

To determine the stresses in the longitudinal steel at cracks, several SR-4 Type A7 electrical strain gages were applied to the reinforcing bars at each of the three preformed cracks as previously described. Additional gages were placed on the steel between cracks to determine the role of the steel in unbroken segments of the concrete. Strain gage data were taken during the experiments in the same sequence as the deflection and crack width data. Strains were multiplied directly by the modulus of elasticity of the steel to obtain the steel stresses.

Significant differences were obtained between the steel stresses at cracks and those removed from cracks; the latter were always quite small—in fact negligible—while the stresses at the cracks were large enough to be critical for the lower percentages of reinforcement.

Variations in steel strains, some as great as 100 percent from the average, were obtained from the several gages at a single crack, under a given load condition. These variations were probably mostly due to eccentricity of forces in the deformed bars resulting from nonsymmetry of bond stresses near cracks. Further contributions to the variations were undoubtedly made by the irregular shape of the cracked transverse sections of the slabs, which caused uneven pressures to develop in the concrete at cracks under vertical loads. It is obvious from these results that one cannot predict exact values of either maximum or average strains or stresses in the steel at a crack; however, it is practically as important to know the order of magnitude of stresses as to know the exact values. Average values do serve as an adequate basis for comparison of the various percentages and positions of reinforcement in arriving at an optimum pavement design.

To obtain representative values of steel stresses at cracks, the measured strains at each of the three preformed cracks were averaged and then the three results were averaged. Fig. 16 shows the influence of temperature drop and percentage reinforcement on the average stress in steel at cracks due to temperature change alone. These curves show an increase in steel stress with increased temperature drops and in general show a definite trend in favor of an increase in percentage of reinforcement, especially at temperature decreases greater than 60 F. Slab 3, with 0.533 percent steel, behaved somewhat irregularly at small temperature drops, but conformed to the general pattern after 60 F change. The position of the steel had considerable influence upon the stresses at cracks due to temperature changes alone. This can be seen in Fig. 17. Slab 5 with steel 2 in. below middepth produced an average steel stress of 18,000 psi at cracks due to 70 F temperature drop, while Slab 6 with the same steel 1 in. higher produced a corresponding stress of 28,500 psi. Middepth placement of the same steel (Slab 3) increased the average steel stress at cracks to 32,000 psi after the temperature drop of

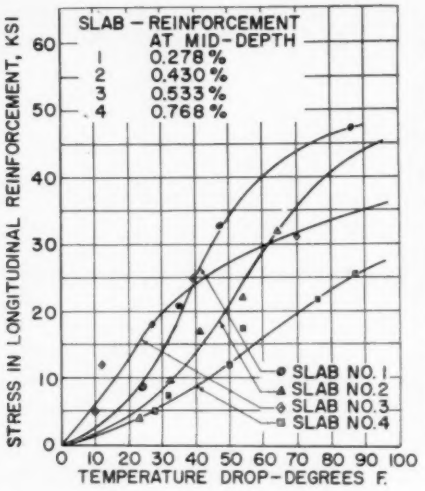

Fig. 16—Influence of temperature drop and percent of reinforcement on stress in steel at cracks

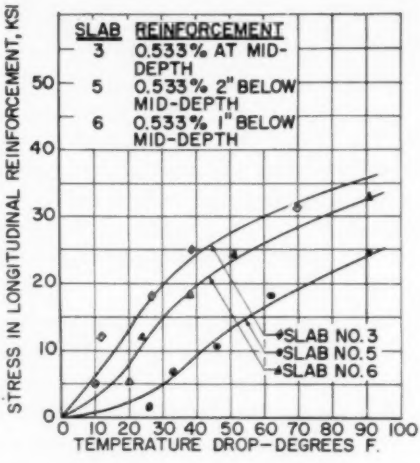

Fig. 17—Influence of temperature drop and position of reinforcement on stress in steel at cracks

70 F. The additional partial cracks formed by the eccentrically placed steel account for the relief of the steel stresses shown by the lowering of the reinforcement.

Vertical loads contribute significantly to the stresses in the reinforcement. Fig. 18 shows the combined effect of 15-kip wheel loads and temperature drops on the steel stresses at cracks for the various percentages of reinforcement. This family of curves shows definitely the inadequacy of the 0.278 and 0.430 percentage reinforcement of Slabs 1 and 2, and casts some question concerning the performance of Slab 3 (0.530 percent steel). The value of the greater amount of steel in Slab 4 (0.768 percent) is quite clear, since its maximum stress was about 35,000 psi, and that occurred after a temperature drop of 100 F. The irregularity and the degree of nonconformity of the stresses of Slab 3 are delineated somewhat by the plots of range of average stress in the steel at the three preformed cracks for Slabs 3 and 4. These ranges are shown by the vertical lines through data points for the two slabs. Slab 4 produced much more regular and more easily interpreted results than Slab 3.

Fig. 19 shows the significant effects of lowering the reinforcement on the stresses produced by combined temperature drops and 15-kip wheel loads. If steel stress were the only criterion, Slab 5 with 0.533 percent steel 2 in. below middepth would be better than Slab 6 with the same amount of steel 1 in. higher. Likewise the latter would be better than Slab 3 with the same steel at middepth. A comparison of Fig. 18 and 19 shows that the steel stresses of Slab 4 (0.768 percent steel at middepth) and Slab 5 were of the

same order of magnitude at any given temperature drop throughout the 100 F range. Slab 6 was stressed to about 40,000 psi at the full temperature drop of 100 F. This reinforcement should be adequate, if some design criterion other than steel stresses (for example—crack widths) should discourage the use of the reinforcement of Slab 5.

CORRELATION OF RESULTS OF LABORATORY EXPERIMENTS WITH REPORTED FIELD TESTS

Field experiments with continuously reinforced pavements have included almost no data on pavement deflections due to wheel loadings. Investigators[13] have experienced difficulties in establishing adequate datum planes for such field measurements, due to the compressibility of considerable depth of subgrade; and the field experiments on continuous reinforcement of pavement slabs have, in general, been directed toward other more pressing problems.

The variation of crack widths due to temperature changes has been investigated in several of the field experiments. After 10 years of service the Indiana (Stilesville) test road[14] showed crack widths from 0.02 to 0.06 in. with an average of 0.038 in. in the lane reinforced with 0.45 percent steel and carrying light traffic. These widths, measured in the fall when the mean pavement temperature was 58-60 F, included raveling and rounding of the concrete edges at cracks. Crack widths measured at middepth on the edge face of the pavement to exclude raveling and rounding showed real widths much less than the upper surface crack widths. These measurements, made with a microscope in the fall when the mean pavement temperature was 73-

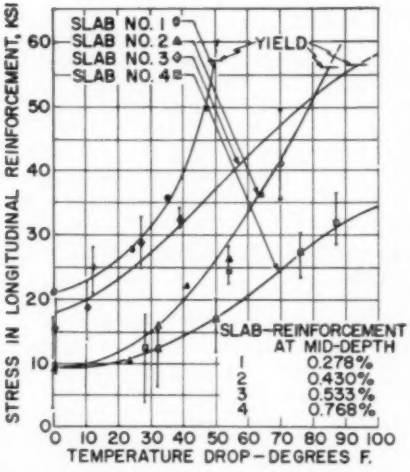

Fig. 18—Influence of temperature drop and percent of reinforcement on stress in steel at cracks due to 15-kip wheel loads

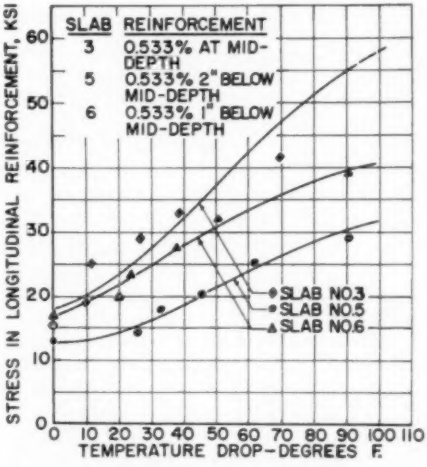
STRESS IN LONGITUDINAL REINFORCEMENT, KSI
SLAB REINFORCEMENT
3 0.533% AT MID-DEPTH
5 0.533% 2" BELOW MID-DEPTH
6 0.533% 1" BELOW MID-DEPTH
SLAB NO.3
SLAB NO.5
SLAB NO.6
0 10 20 30 40 50 60 70 80 90 100 110
TEMPERATURE DROP-DEGREES F.

Fig. 19—Influence of temperature drop and position of reinforcement on stress in steel at cracks due to 15-kip wheel loads

74 F, showed crack widths between 0.007 and 0.010 in. in the same section of pavement mentioned above. Similar data for other percentages of reinforcement showed that the crack widths, for a given temperature, increased with a decrease in the percentage of longitudinal reinforcement.

The 3-year performance report on the continuously reinforced pavement in Illinois[15] showed that 7- and 8-in. slabs reinforced with 0.3, 0.5, 0.7, and 1.0 percent steel had developed transverse cracks ranging in width from 0.007 to 0.021 in. by August, 1950. After 3 years of service from the New Jersey[16] experimental pavement, measurements showed a maximum crack width of 0.019 in. and an average crack width of 0.015 in. in the outside lane of an 8-in. pavement, at a net temperature drop of 31 F. This pavement was longitudinally reinforced with 0.90 percent steel.

Thus it is seen that crack widths obtained in the laboratory were of the same order of magnitude as those observed in the field.

Changes in crack widths with passage of wheel loads have not been measured in field experiments. Thus no correlation can be made between laboratory and field data in this important phase of the laboratory experiments.

Steel stresses have been measured in the Illinois,[15] California,[17] and Pennsylvania[4] experiments. Stresses as high as 62,000 and 42,000 psi were reported for the Illinois experiment for 7- and 8-in. slabs reinforced with 0.7 percent steel. Stresses as great as 50,000 and 80,000 psi were reported for the California experiment 6 months after casting the concrete in 8-in. slabs reinforced by 0.62 and 0.50 percent steel, respectively. Stresses above the yield strength of the steel were reported 2 months and 3 months after casting the York, Pa., pavement containing 0.50 percent longitudinal steel.

The California report[17] on steel stresses stated:

> "At early ages (less than 3 months) the readings on pairs of SR-4 gages opposite each other on the same reinforcing bar were in agreement within 10 percent. There was, however, a noticeable increase in the variations in readings at 3 months and at 6 months."

As a result of the laboratory experiment, it is believed that such variations are to be expected as a result of unequal stress build-up due to temperature changes and wheel loads rather than as a direct result of the passage of time.

TABLE 2—EVALUATION OF SLABS IN TERMS OF SUGGESTED DESIGN CRITERIA

Slab No.	Percent reinforcement	Position of reinforcement	Rating* according to criteria			
			Vertical movement	Crack width	Steel stress	Crack formation
1	0.278	Middepth	C	D	D	D
2	0.430	Middepth	C	D	D	D
3	0.533	Middepth	C	C	C	A
4	0.768	Middepth	B	B	A	A
5	0.533	2 in. below middepth	A	D	A	B†
6	0.533	1 in. below middepth	B	A	B	A†

*RATING DEFINITIONS: A—satisfies criterion completely.
B—satisfies criterion nearly completely in comparison with other specimens.
C—satisfies criterion partially, less than other specimens.
D—criterion rules out the design completely.

†Criterion not yet intended to apply to slab reinforcement placed other than at middepth.

The several field experiments yielded varied crack widths and stresses in the various types of reinforcement due to temperature changes. These variations can be explained through consideration of influencing variables such as subgrade stiffness, concrete casting temperatures, and subsequent moisture conditions. The crack widths and steel strains reported for the field experiments were of the same order of magnitude as those obtained in these laboratory experiments.

Data on crack widths and steel stresses due to wheel loads as well as temperature changes have not been reported from the several experiments.

DESIGN CRITERIA

A study of laboratory data on vertical deflections, crack widths, and reinforcing steel stresses, on an individual basis, suggests the following criteria for the design of continuously reinforced pavements:

1. For the temperature range between the casting temperature of the concrete and the freezing temperature of the subgrade, the total vertical movement composed of downward deflection under wheel loads and upheaval as the loads move away should be minimized as nearly as practicable.
2. For the same practical temperature range delineated in Criterion 1, the upper surface and lower surface crack widths should be considered equally important and the maximum active crack widths due to wheel loads as well as temperature changes at both the upper and lower surface should be minimized as nearly as practicable.
3. For the practical temperature range, the maximum reinforcement stress (average) due to temperature changes and wheel loads should be maintained as far below the yield strength of the steel as is economically feasible.

In general, all three of these design criteria cannot be completely satisfied by a single choice of amount and position of reinforcement. Furthermore, economic considerations may place practical restrictions on the designer in his choice of concrete thickness and steel reinforcement for continuous pavements which might make these criteria unattainable; thus the criteria must serve as a guide for compromise in the choice of design characteristics of continuously reinforced pavements.

A previous study[6] of formation of cracks in continuous pavements has suggested a fourth criterion which should be satisfied along with the three listed above. It can be summarized thus:

4. Sufficient longitudinal reinforcing steel must be provided to maintain all transverse cracks tightly closed. For middepth reinforcement, this condition is met when increasing longitudinal forces caused by temperature drops, in combination with live loads, tend to cause additional cracks rather than to open existing cracks increasingly wider.

One can apply these four criteria in judging the relative merits and acceptability of the six slabs included in the laboratory experiments. It must be remembered, however, that the experiments to date have included only slabs 8 in. thick resting on a subgrade having a modulus of 160 lb per cu in. Thus optimum design for a given field condition may not be represented among

the slabs reported here. Table 2 summarizes the application of the four criteria to the slabs tested.

Table 2 reveals the general inadequacy of the reinforcement in Slabs 1 and 2; the inadequacy of Slab 5 with its reinforcement 2 in. below middepth, because of excessive upper surface crack widths; the advantage of 0.533 percent steel 1 in. below middepth (Slab 6) over the same reinforcement at middepth (Slab 3); and the somewhat equal performance of Slab 4 (0.768 percent steel at middepth) and Slab 6 (0.533 percent steel 1 in. below middepth). The economic advantage of Slab 6 over Slab 4 would make its choice certain.

Thus of the six specimens tested to simulate 8 in. thick continuously reinforced concrete pavements resting on subgrades having a modulus of 160 lb per cu in., reinforcement with 0.533 percent steel placed 1 in. below middepth appears to represent the most economical design for best performance.

SUMMARY OF OBSERVATIONS DRAWN FROM LABORATORY RESEARCH

As a result of this study of deflections, crack widths, and reinforcement stresses in continuous pavements reinforced with deformed bars, and within the limitations imposed by the range of variables studied, the following observations are made:

1. Decreases of about 20 F from the temperature of casting a pavement, continuously reinforced with deformed bars, tend to *reduce* slightly the slab deflections due to vertical loads. Temperature decreases greater than 30 F tend to *increase* the deflections due to vertical loads.

2. Percentage of middepth reinforcement influences maximum deflections due to vertical loads; maximum deflections vary somewhat inversely as the percentage of reinforcement.

3. The position of the deformed bar longitudinal reinforcement has a definite effect on the live load deflections of the pavement. As the reinforcement is lowered, deflections are decreased.

4. Upper surface crack widths vary somewhat linearly with temperature decreases in underreinforced slabs; pavements with adequate reinforcement form new cracks during sizable temperature decreases, and old cracks do not continue to widen in direct proportion to the temperature drop.

5. Reinforcement below middepth in the pavement causes partial cracks to be formed in the lower portion of the slab with fewer complete cracks extending to the upper surface. The upper surface control of the crack widths due to temperature changes alone is not the best when the reinforcement is placed as low as one-fourth of the depth above the bottom of the slab.

6. Maximum active crack widths (due to temperature drops and wheel loads) at the upper and lower surfaces of the slab can be equalized and minimized by proper placement of reinforcement. Reinforcement of 0.533 percent 1 in. below middepth is better in controlling top and bottom active crack widths than 0.768 percent steel at middepth.

7. Significant differences exist between the longitudinal steel stresses at cracks and those removed from cracks; the latter are always quite small while the stresses at cracks tend to be a critical indication of the adequacy of the reinforcement.

8. Variations in steel strains (and stresses), some as great as 100 percent, exist among the several gage locations on the steel at a single crack, under given sizable temperature changes and live loads.

9. An increase in average steel stress at cracks accompanies increased temperature drops (below concrete casting temperature); the stresses vary somewhat inversely with percentage of longitudinal reinforcement at middepth.

10. Position of reinforcement influences the steel stresses at cracks due to temperature drops alone. Moving the steel below middepth decreases the steel stresses in a somewhat linear relationship to the distance below middepth.

11. Vertical loads contribute significantly to steel stresses.

12. Increasing the percentage of steel placed at middepth in the pavement decreases the steel stress at cracks due to a given temperature decrease and live load.

13. Moving the reinforcement below middepth decreases the steel stresses due to a given temperature change and wheel load, somewhat linearly with the distance of the steel below middepth.

14. Steel stresses, due to temperature drops and a 15-kip wheel load, in a slab reinforced with 0.768 percent steel at middepth are about the same as those in a slab with 0.533 percent steel 2 in. below middepth.

15. Application of the four suggested design criteria (pp. 243-244) shows that Slab 6 simulating a continuous pavement reinforced with 0.533 percent steel placed 1 in. below middepth was one of the two best and was the most economical of the six slabs 8 in. thick tested on a subgrade having a modulus of 160 lb per cu in.

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Title No. 56-17

Precast Units for New Aluminum Plant

By ROSS H. BRYAN

About 8000 precast, prestressed concrete units, including columns, brackets, beams, and floor slabs, were used in this project. Several intricate floor slab shapes were required, containing ventilation holes. The project covered an area of about 980 x 1075 ft.

Connections between all units, including columns, were made by bonding reinforcing bars into grouted cavities; welded connections were not permitted because of high voltage lines carried on the column brackets.

■ THE NEW ALUMINUM PLANT of the Reynolds Metals Co. at Listerhill, Ala., cost approximately $70,000,000 and will produce 200,000,000 lb of aluminum per year.

Part of the plant's production is molten aluminum supplied to a nearby casting plant of the Ford Motor Co. The construction timetable was keyed to the operation date of the Ford plant, which was being built concurrently. The simultaneous construction of the two plants required a positive and workable construction schedule to make certain that a supply of aluminum would be available to the Ford facility when it was ready to begin operation.

To meet the required construction schedule, Reynolds' engineering division decided to use precast, prestressed concrete for the operating floors and roof of the rectifier building and the operating floors of the nine pot room buildings. The plan was to fabricate the floor and wall elements while foundations were being placed and while awaiting delivery and erection of the structural steel frames supporting the roofs and craneways. The idea proved workable in the field and the precast concrete elements were stockpiled and erected without interruption of the steel erection schedule. It was also decided that precasting would be more economical than cast-in-place construction due to the intricate shapes of the slabs and their requisite ventilating holes. Other considerations were minimum construction depth, which could be obtained only by

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prestressing, and the time schedule. The decision was based on the judgment of several people experienced in both cast-in-place and precast construction. No alternate designs were made or estimated.

The over-all project included one rectifier building with a ground floor and one framed floor, 32 ft wide and 800 ft long, and 18 pot rooms, 67½ ft wide by 420 ft long with interconnecting passageways. Fig. 1 shows an over-all

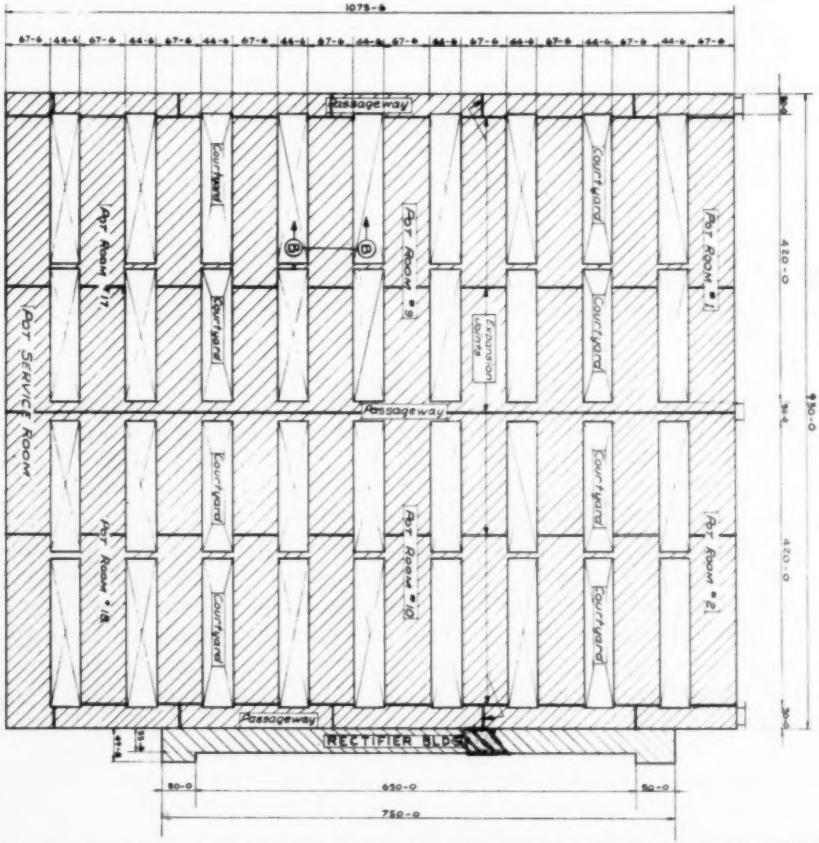

Fig. 1—Plan view of project indicating pot rooms, courtyards, and expansion joints (Section B shown in Fig. 4)

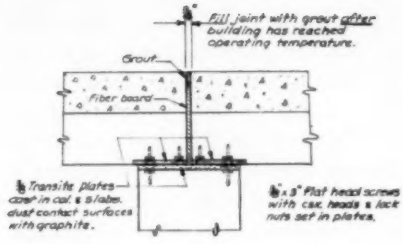
Fill joint with grout after building has reached operating temperature.
Grout
Fiber board
Transite plates—cast in col. & slabs. dust contact surfaces with graphite.
x 3" Flat head screws with cas. heads & lock nuts set in plates.

Fig. 2—Details of expansion joint used in floor slabs

plan of the project and the location of the expansion joints. The building area is indicated by the crosshatched sections, with the open spaces indicating courtyards. The pot room buildings also have a ground floor and one framed operating floor.

Fig. 2 shows the details of the expansion joint used in the pot rooms. This type of joint construction had been used on bridges prior to its use on this project and was found to be an efficient and economical detail. Its performance in the building so far has been satisfactory.

The floor construction of the rectifier building was rather commonplace, as far as the precast concrete elements were concerned, consisting of standard channel slabs supported on steel framework. The exterior of the rectifier building was somewhat unusual in that it consisted entirely of precast, prestressed panels enclosing the sides and ends of the building to a height of 45 ft without windows. Some of these panels required many openings for bus bars and other utilities. The panels were fastened to steel frames. The longitudinal joints were tongue and groove and sealed with a precast rubber strip. The vertical joints were closed with an aluminum strip which was also sealed with precast rubber strips (Fig. 3).

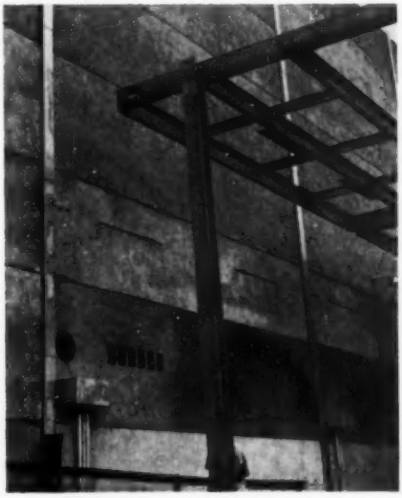

Fig. 3—Aluminum strips used as vertical joints between precast, prestressed end and side panels

The design of the operating floor of the pot room buildings was unusual in that the members required to fit around the ore reduction pots were quite complicated in shape and, in addition, required the forming of ventilating holes in each panel. These holes were $2\frac{1}{2}$ in. in diameter and spaced 6 in. on center in each direction. Headroom was placed at a premium and the over-all depth of the construction was held to a maximum of $19\frac{1}{2}$ in.

The original design concept required that no placing of structural concrete be done above the foundation floor slabs. This meant that all structural elements, together with their supports and brackets, had to be precast. The structural system of the operating floor consisted of two separate framing conditions: the supporting beams and columns for the pots, and the operating floor system and passageways surrounding the pots.

The operating floor system included two general types of precast panels. One was a single-T member with one edge of the slab supported on an exterior steel spandrel beam. The second member was a double-T slab covering the space between the pot lines. Each of these slabs was supported on precast columns located under the stems of the T-sections. There were no transverse beams used in the floor framing. Fig. 4 is a typical section through the pot rooms.

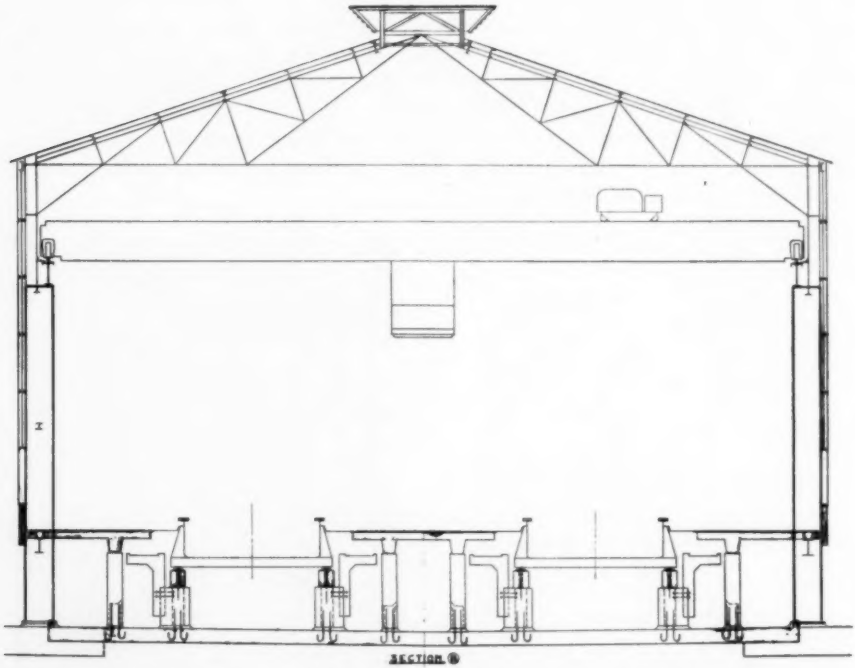

Fig. 4—Typical cross section of pot rooms showing structural features (Section B of Fig. 1)

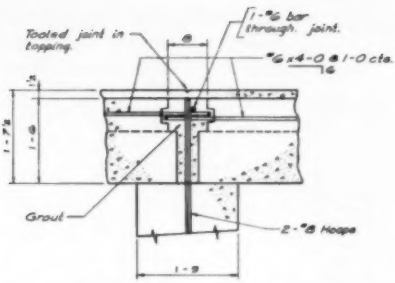

Fig. 5—Detail of connection between two operating floor slabs showing the keyway, projecting reinforcing steel, and supporting column

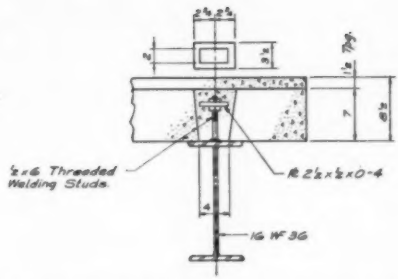

Fig. 6—Detail of connection between two single-T slabs. Cutouts in slab allow welding of steel bolt to spandrel beam, this cavity is later grouted

The span of the floor slabs is 14 ft center to center of columns and they are designed to support a moving group of four live loads of 18,000 lb each. Diagonal tension in the slab due to these concentrated loads was not a factor in the design. The 18,000 lb wheel loads are no greater than those used in the design of highway slabs and the slab thickness of 7 in. is that normally used on highway construction. These loads are located at the corners of a 5-ft square and are applied when the pots are removed from their supports for overhaul and repair.

The stems of all T-sections were prestressed to carry the longitudinal bending moments. The slabs of the T-sections are reinforced transversely with mild steel and considerable distribution steel was used in a longitudinal direction to distribute the concentrated loads from the pot supports.

The columns supporting the operating floor slabs were fastened to the foundation by reinforcing steel extending from a footing into a grout cavity formed in the bottom of the column. The floor slabs are seated on top of the column and spaced so that reinforcing steel extending from the column into the space forms a tie when the space is filled with concrete. The ends of the slabs are keyed and reinforcing steel projects from the keyway of each slab into an 8 in. wide strip which was formed and cast in the field to form a joint which would transfer and distribute the concentrated loads from the pots, in the event a support was placed directly over the joint in the web of the slab (Fig. 5).

Where the slab of the single-T is supported on an exterior steel spandrel beam, cutouts were formed in the slab so that a steel bolt could be welded to the top of the beam after the slab was in place. The cavity around this bolt was later filled with concrete to form a tie (Fig. 6).

The floor slabs were of four general types and their intricate shapes warrant some attention. Fig. 7 shows the slab which was placed between the pot lines. This slab has many complicated cutouts in plan, dap outs to clear the electrodes of the pot frames and holes at 6 in. centers in each direction

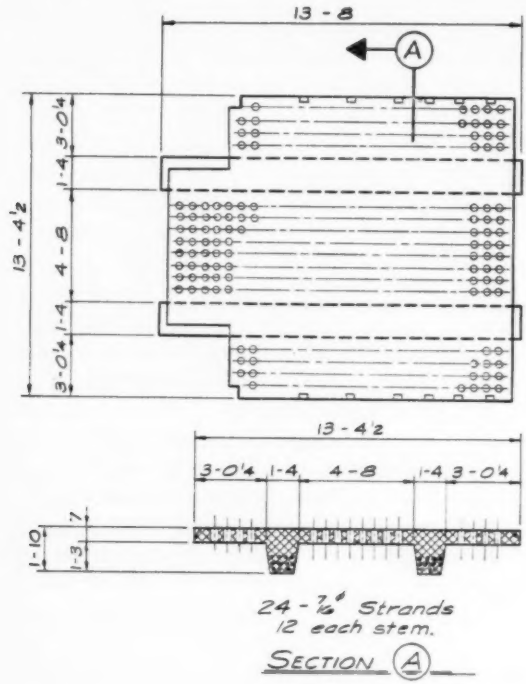

Fig. 7—Floor slab used in pot rooms, ventilating holes and other features (cutouts and dap outs) are shown

for ventilating the lower area. There were approximately 550 slabs required of this type. The ventilating holes were formed in the slab by means of rubber mandrels bolted to the form (Fig. 8).

Fig. 9 shows the slab required between the pot line and the exterior of the building. The long overhang of the slab of this section is supported by the

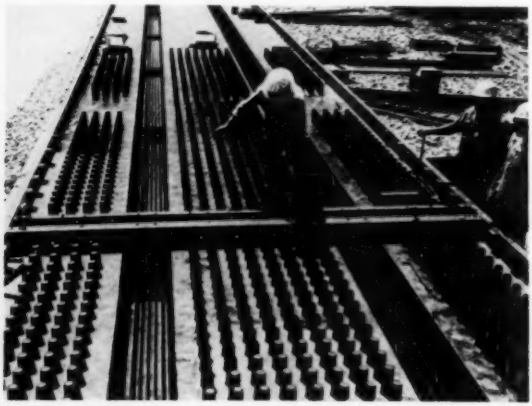

Fig. 8 — Rubber mandrels bolted to slab to form ventilating holes for slabs between pot lines

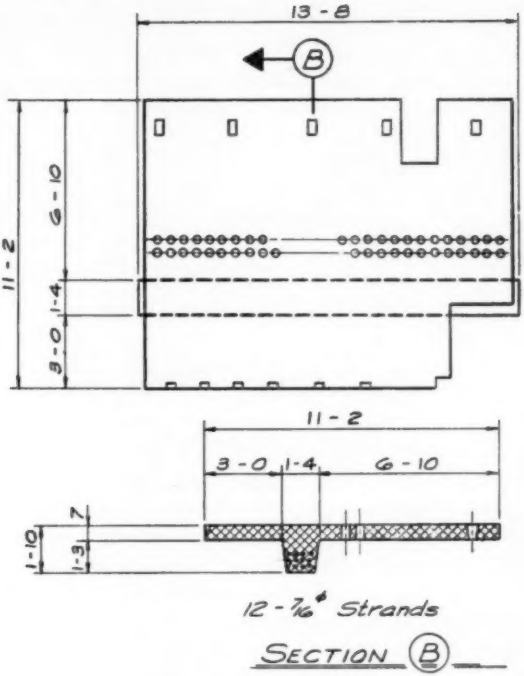

Fig. 9—Slab required between pot line and exterior of building. Long overhang is supported by steel spandrel beam as shown in Fig. 6

steel spandrel beam shown in Fig. 6. There were approximately 1100 slabs required of this type.

Fig. 10 shows a slab which was placed between the ends of the pots. It is not only quite complicated in shape but it will be noted that the stem of this slab is cut short of the support to clear the bus bars supplying electricity to the pots. The ends of the slab section were specially reinforced to transmit the shear and moment at the support. There were approximately 560 slabs required of this type.

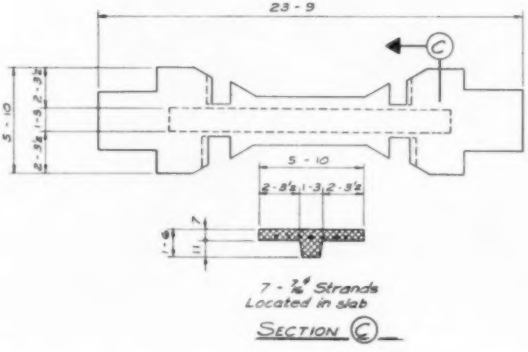

Fig. 10—Diagram of closure slab placed between pot ends

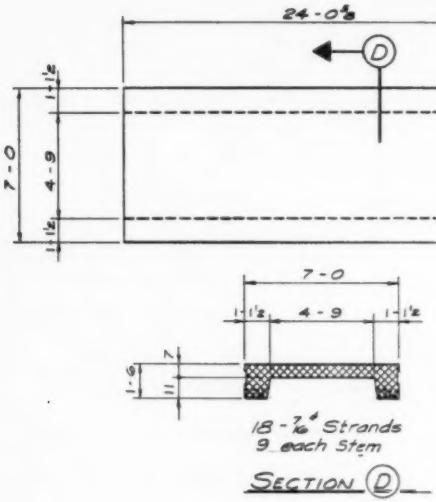

Fig. 11—Typical channel slab used in passageways

The slab shown in Fig. 11 is a typical channel slab and was used principally in the passageways. There were approximately 700 slabs of this type required. Fig. 12 shows master casting bed used to cast all of the floor slabs.

There were no special problems encountered during erection which involved misfits or required drifting operations. Sufficient space was allowed between the ends of members and around the sides of the pots and walls to permit slight variations in dimensions without resulting in a clearance problem.

The reduction pots were supported on two sides by prestressed, precast beams which span 14 ft and cantilever 4½ ft on either end. The loading on the beams was somewhat unusual and consisted of a total load of 100,000 lb for each beam, distributed in the shape of a parabola. The curvature of

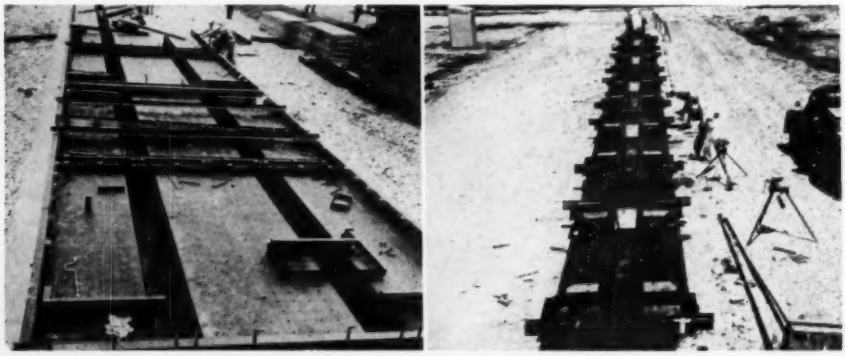

Fig. 12—Master casting bed (left) used to cast all of the variously shaped floor slabs. Holes in base were used to fasten rubber mandrels for ventilation openings. (Right) Master bed with fillers and blockouts used for closure slabs between pot ends

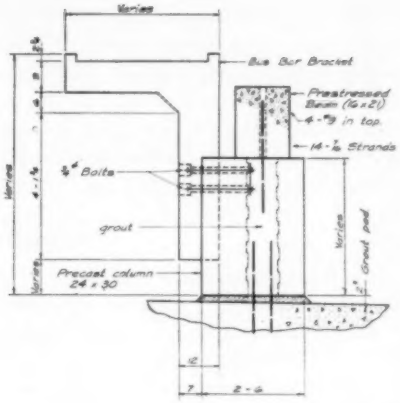

Fig. 13—Details of pot beams columns and bus bar brackets

the parabola could be either concave with maximum load ordinate at the ends of the beams reducing to zero at the center, or it could be convex with a minimum ordinate at the center of the beam reducing to a zero at the ends. This unusual loading condition is brought about by expansion and contraction of the pots as they are heated and cooled. In designing the pot beams, it was decided to reinforce with mild steel for the negative cantilever moment over the support and to use prestressing strands to resist the positive moment.

The pot beams were supported by precast columns which were anchored to the foundation by reinforcing steel extended from the footing into a cavity formed in the column. The pot beam was fastened to the columns by extending a reinforcing bar through a sleeve in the beam into the grout cavity of the column below. Brackets to support the bus bars were precast and bolted to the column after the column had been placed. Fig. 13 shows the details of the pot beam columns and bus bar brackets. There were approximately 2350 brackets and pot beam columns required, and 1064 pot beams were used on the project.

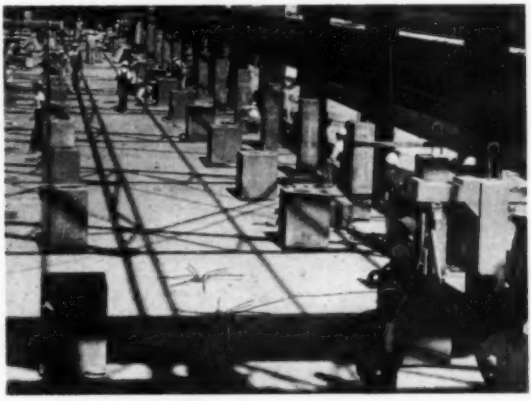

Fig. 14—Typical pot room building during construction showing short, cored columns that support pot beams. Floor columns are at right

Fig. 14 shows an interior view of a typical pot room building during construction. The short, cored columns support the pot beams. The slots in these columns provided for the bus bar brackets may also be seen. The star pattern of reinforcing spread out on the floor slab will be bent up into the grout cavities of the columns supporting the floor slabs. Floor slab columns have been placed at the right. The reinforcing tie at the top of the columns extends into the space between the ends of the floor slabs.

ACKNOWLEDGMENT

The precast members of the rectifier building were manufactured in the plant of the Concrete Engineering Co., under the direction of William Baskerville.

The precast members of the pot buildings were fabricated on the site by the Texstar Corp., San Antonio, under the direction of James J. Mennis.

Engineering and architectural design work was done by Day and Zimmerman Co., Philadelphia, under the supervision of Reynolds Metals Co. engineering department.

The design details of the precast members for the rectifier and pot room buildings were made in the office of the author, under the direction of Robert L. Whitaker.

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Title No. 56-18

SULFATE ATTACK ON CONCRETE IN THE OSLO REGION

JOHAN MOUM and I. TH. ROSENQVIST

In the Oslo region of Norway, alum shales* containing small amounts of the unstable iron sulfide, pyrrhotite, produce an unusual form of sulfate attack upon concrete placed in or near these deposits, and cause deterioration if they are used as concrete aggregate. The ground water associated with the alum shales carries ferrous sulfate and produces severe sulfate attack and the precipitation of ferric iron compounds in concrete structures made with normal portland cement. Cements of low tricalcium aluminate content resist the sulfate attack but may be subject to attack by acid solutions produced when the ferrous sulfate is oxidized. Air-entrained concrete appears to be particularly susceptible.

■ FOR 40 YEARS THE CONSTRUCTION INDUSTRY in the Oslo region has been plagued with problems of concrete deterioration and foundation upheaval related to the presence of slightly metamorphosed shales containing an unusually unstable form of the iron sulfide mineral pyrrhotite. The shales are called "alum shales"* or "alum slates," and the expression "the alum slate problem" is familiar to most people engaged in construction work in Norway.

After World War II a semiofficial "Alum Shale Committee" was set up in Oslo, and the Norwegian Geotechnical Institute was requested to take over problems especially connected with the chemical, physical, and mineralogical phases of the work. Some of the results of work on this unusual form of sulfate attack are summarized below from previous papers[1,2] and subsequent investigations.

Geology

The middle Cambrian–lower Ordovician shales and slates of the Oslo region represent the moderately metamorphic equivalent of sediments of high sulfur and carbon content. Comparable sediments of varying degrees of metamorphism are widely distributed in Scandinavia, ranging from bituminous shales in central Sweden, to anthracite shales and slates in the Oslo area, to graphitic slates in northern Norway. In the Oslo region the carbon content averages around 7 percent and the sulfur content around 6 percent, but both values

*The term "alum shale" originated from the fact that this rock type in Scandinavia was used for manufacturing potassium aluminum sulfate, $KAl(SO_4)_2 \cdot 12\ H_2O$.

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vary widely. The alum shales also contain as much as 30 to 200 g of uranium per ton. Similar shales are known in many regions; the Chattanooga shale of the eastern United States is one example.

SULFIDE MINERALS

In the unmetamorphosed shales of central Sweden the sulfur occurs as pyrite (FeS_2), but in the Oslo area part of the sulfur is present as monoclinic pyrrhotite. This pyrrhotite has the composition $FeS_{1.14}$, and the x-ray pattern corresponds to that of synthetic monoclinic pyrrhotite.[3-10]

The reactive natural pyrrhotite can only be kept a short while in moist air. During crushing of the natural rock and subsequent preparation, the pyrrhotite is most often lost and only pyrite is left, which explains why pyrrhotite was not identified earlier. By very careful preparation, in the absence of air, the latter mineral has been concentrated and determined by x-ray diffraction.

The pyrrhotite content is related to the rate of alteration of the pyrite, to the rate of weathering of the shale, and to the aggressiveness of the shale to portland cement concrete.

All shales which weather "explosively" in the presence of air have been found to have a content of more than 0.2 percent sulfur in the form of monoclinic pyrrhotite. The pyrrhotite content is elsewhere normally fairly low and the amount of sulfate formed during the weathering processes is higher than that corresponding to the sulfur present as pyrrhotite. In these cases it cannot be doubted that part of the sulfate originated through oxidation of the pyrite. There is a general tendency toward the formation of more sulfate in the rocks containing much pyrrhotite than in the rocks containing little or no pyrrhotite. Thus it seems that the pyrrhotite in some way accelerates the oxidation of some pyrite. Microscopic investigations of the unweathered alum shale have shown that a great part of the pyrrhotite is present in minute grains often in direct contact with the pyrite. Both these minerals are electrical conductors. Mixtures of sulfide minerals oxidize more quickly than when the single minerals occur separately.[5] Similar processes are found when pyrite and pyrrhotite are in electrical contact.

As mentioned above, there seems to be a relation between the presence of monoclinic pyrrhotite and the reactivity of the alum shale. We have

never found important damage or concrete deterioration when the alum shale has been free from pyrrhotite, and the intensity of damage seems to increase with increasing pyrrhotite content.

WEATHERING PRODUCTS

The weathered alum shales are mostly covered by a yellow deposit of jarosite [$KFe_3(OH)_6(SO_4)_2$] and brown-iron ore ($Fe_2O_3 \cdot nH_2O$).

The weathering of the alum shale also yields solutions which very rapidly attack concrete made with normal portland cement. We have seen the concrete walls of an underground bomb shelter built in an alum shale area transformed into mush in about 9 months. In other cases, the attack may proceed more slowly, but generally the attack from the alum shale extracts seem to be much quicker than attack by most other aggressive waters.

It has been believed that the water percolating through a weathering alum shale is very acid. In one locality the pH of the water was reported to be about 2.5. As this value seemed to be extremely low, we took new samples, prevented air from coming in contact with the water, and performed a complete analysis without any secondary oxidation (Table 1). These samples of water were taken from "Blindtarmen," a tunnel in the Oslo alum shale used as a field laboratory by the Alum Shale Committee. The tunnel was intended to carry sewage, but because of the aggression of the alum shale upon the concrete lining, and other local conditions, this part of the sewer system was closed off and never used.

The marked discrepancy between our analyses and earlier analyses needs an explanation. The water does not contain trivalent iron, but fairly large amounts of bivalent iron together with sulfates. When bivalent iron sulfate is secondarily oxidized, the equation of the reaction may be written:

$$2FeSO_4 + 5H_2O + 1/2\ O_2 = 2Fe(OH)_3 + 2H_2SO_4$$

If, for instance, 1 g of ferrous sulfate is dissolved in a liter of water, the solution will have a pH of 5.42. After 1 min of shaking in air, the pH will drop to about 4, and on further mixing with air drops to 2.8, about the pH value found for diluted ferric sulfate solutions, and near the pH value previously found in the Blindtarmen water.

TABLE 1—ANALYSIS OF WATER PERCOLATING THROUGH ALUM SHALE AT BLINDTARMEN

Impurity measured	Sample I	Sample II	Sample III	Sample IV
Residue 105 C, g per l	5.38	6.14	5.00	4.89
Al	trace	trace	trace	trace
Fe+++	trace	trace	trace	trace
Fe++, g per l	0.358	0.412	0.244	0.229
SO_3, g per l	2.3136	0.544	2.109	2.104
Cl, g per l	0.421	0.491	0.343	0.347
pH	6.22	5.28	6.40	6.02

Fig. 1—Ettringite seams around alum shale in concrete

Normally the solutions from the alum shales are much higher in sulfate than would be expected as a result of the oxygen content of normal rain water. Therefore, we have to assume that the oxidation of the solutions takes place above the water table where the pore space of the rock is partially saturated with water and partially filled with air. The sulfate-bearing solution may penetrate fairly deeply into the rock, and the aggressive solution is found at considerable depths.

CONCRETE DETERIORATION

The alum shale has caused complete deterioration of concrete structures in a very short time. The Alum Shale Committee started its experiments in the Blindtarmen tunnel where water entering through the destroyed lining comes directly from one of the most aggressive zones in the alum shale. The tunnel was partially full of water, with the water level varying somewhat according to precipitation. Air was allowed to enter above the water through a shaft up to the surface. The water in the upper part was therefore oxidized through the air above, whereas in the deeper part the water corresponded to the analysis given previously.

The Alum Shale Committee placed some of its concrete prisms deep in the water below the low-water level, and some between the low-water and the high-water level. After 3 to 4 years the prisms left above the low-water level still had sharp edges and fairly good mechanical properties. However, those left deep in the tunnel were mostly destroyed as reported by Fiskaa.[2] All the specimens located at depth had suffered somewhat except specimens containing sulfate resistant cement. In the upper specimens no such marked difference was seen; all were covered by a brown layer of rust. The specimens from the bottom were black or greenish and had obviously undergone swelling processes in the surface layers. Large amounts of white pulverized material was seen, which chemical and mineralogical investigations (differential thermal analysis and x-ray and microscopic investigations) indicated to consist mainly of ettringite; no gypsum or anhydrite was found. Thus, the product in the deteriorated concrete is the same as normally found in con-

crete exposed to sulfate attack. Ettringite has also been found in concrete where (by some error) alum shale has been used as aggregate (Fig. 1). In such cases aggregate swelling has caused cracks and destruction of the concrete.

As ettringite is normally formed by sulfate attack, the final result of the alum shale and normal sulfate attack are the same but there is a great difference in the velocity of attack. Normal sulfate attack may take several years to cause any noticeable damage, whereas the water from the alum shale containing ferrous sulfate has caused considerable deterioration in very few months. Even the best concrete will not stand more than a few years if portland cement is used, whereas sulfate resistant cement seems to stand very well. This again proves that it is not an acid attack we have to deal with, but some peculiar sulfate attack.

Mechanism of attack

The mechanism of attack seems to be closely connected with the composition of sulfate solution entering into the system. In alum shale the solution contains bivalent iron, which is later more or less oxidized to trivalent iron. In the samples from the upper part of the tunnel the conditions were different. The water there was acid and contained trivalent iron. There are differences in the types of attack, even though the sulfate ion content is the same, in the three instances of sulfuric acid, sodium sulfate, and bivalent iron sulfate attack on concrete. Sulfuric acid mainly attacks the surface of the concrete, dissolves the cement gel and paste here, and forms gypsum from the calcium hydroxide and other components easily attacked. The solution which penetrates further into the concrete contains no more sulfate than the amount in a saturated gypsum solution, about 800 mg SO_3 per l, or about one-third of the sulfate concentration in the Blindtarmen water. Eventually trivalent iron will be precipitated upon the surface of the concrete under such conditions, and fill the pores and make the concrete more watertight, so that further penetration will be slower.

In contrast, sodium sulfate solution may penetrate into the concrete without immediate reactions at the surface, and the quantities of sulfate brought into the concrete may be much higher. In the interior of the concrete these amounts of sulfate will cause reactions with the cement. The reactions may form either sodium hydroxide or sodium carbonate and due to this the pH will increase from 10 to 13, which seems to counteract the formation of the aluminate sulfate, ettringite.

In contrast to these two types of attack, bivalent iron sulfate may penetrate the sound concrete in the same way as the sodium sulfate solutions and carry with it higher concentrations of sulfate. In the interior of the concrete it will precipitate insoluble iron hydroxides which may oxidize, and we will have no increase in the pH, but more probably a decrease. Thus, we have to deal with a combination of two different types of attacks, a typical sulfate attack and an acid attack. Simple acid attack will, however, take place at

pH values lower than about 4, whereas sulfate attack in one form or another may take place at higher pH values. Fig. 2 shows the solubility curve for bivalent and trivalent iron as a function of pH. Trivalent iron is only present at low pH values, and if solutions containing bivalent iron are oxidized, the pH will drop correspondingly, precipitating trivalent iron.

The attack of bivalent iron sulfate solutions on concrete is similar to what we may have with a combined sulfate and aggressive carbon dioxide attack.

The influence of carbon dioxide upon concrete has been dealt with by Ruth Terzaghi, (1949)[11] and a combined effect by G. M. Idorn (1953)[12]. In the case of iron sulfate solutions we may say that the iron sulfate will have a "carbon-dioxide equivalent." This means that the buffer capacity of bivalent iron will decrease the pH and act in the same way as if we had the sulfate ions together with aggressive carbon dioxide, (i.e. a content of carbon dioxide higher than corresponding to equilibrium between calcium carbonate and carbon dioxide on one side, and calcium bicarbonate on the other side). Fig. 3 shows the "carbonate equivalent" of the Blindarmen water; under such conditions we will have a much stronger and quicker action of the sulfate upon concretes containing tricalcium aluminate, i.e. normal portland cement. With such buffered systems, we may have a primary precipitation of ettringite and on a subsequent oxidation a further decrease in the pH, followed by a re-solution of the sulfate from the ettringite. The sulfate may act several times, steadily destroying the concrete.

Air-entrained concrete

The alum shale solutions seem to attack concrete containing microscopic air bubbles more quickly than other concretes. Bivalent iron will react with air, especially under fairly alkaline conditions, thus depleting the air bubbles of their content of oxygen. As normal air contains more than 20 percent by

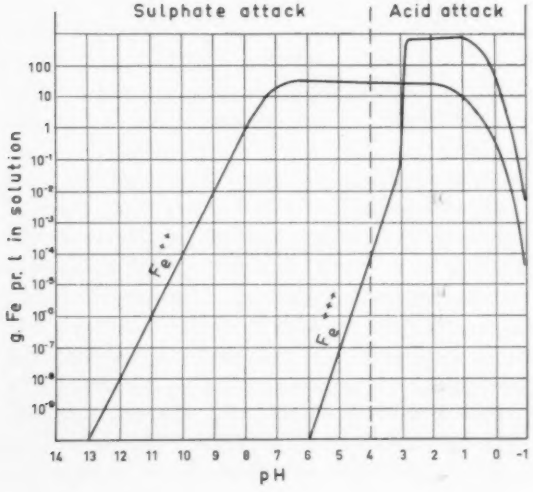

Fig. 2—Solubility of trivalent and bivalent iron sulfate as a function of pH, and the corresponding attacks on concrete

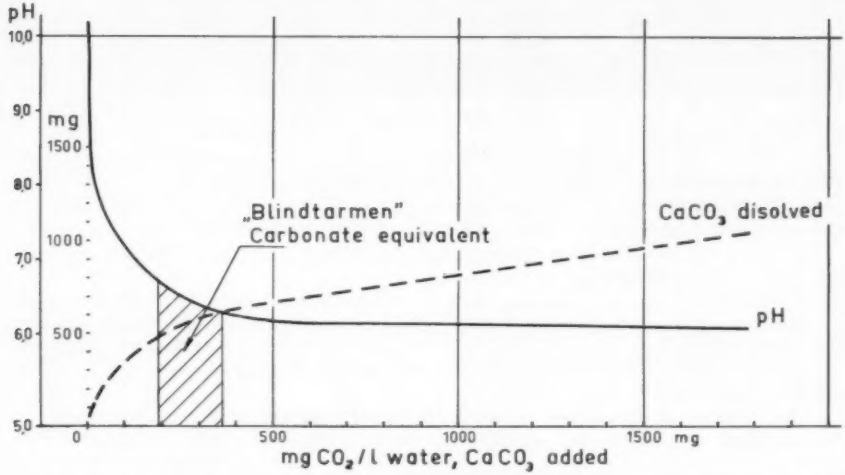

Fig. 3—Carbon dioxide equivalent (i.e., the equivalent buffer capacity due to Fe++) of aggressive water from Blindtarmen test tunnel

volume of oxygen, such a reaction between bivalent iron solution entering into one pore will cause a negative pressure in the pore of 200 g per sq cm, and if the distance from the surface to the pore is only of the order of a few tenths of millimeters, reduced pressure in the pore will cause suction and hydraulic gradient of the order of magnitude 1000 in the outermost concrete, so that the pores will act as vacuum pumps and suck in the aggressive solution from the surroundings. Thus, the aggressive solution may be sucked through even by fairly impervious concrete and ettringite formed. Here again is a typical difference between a bivalent iron sulfate solution and other normal sulfate solutions which may be found in nature.

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CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Erection study of prestressed bridges

The Engineer (London), V. 206, No. 5347, July 18, 1958, p. 81 and p. 92

Economics of precast prestressed bridge construction

Engineering (London), V. 186, No. 4820, July 25, 1958, p. 127

Reviewed by Aron L. Mirsky

All-too-brief descriptions (with editorial comment) on a demonstration, investigation, and training area installed by one manufacturer. Several precast bridges and one cast-in-place structure have been constructed, to study methods, time, costs, of spans up to 125 ft, using various systems.

Moment analysis for bridge abutment wingwalls of variable thickness

C. L. Hulsbos and Ti-Ta Lee, *Bulletin* No. 184, Iowa Engineering Experiment Station, May 1959, 36 pp., $0.75

Part of a series on bridge abutment wingwalls of the flared type (*Bulletin* No. 182 and 183 were reviewed in the July ACI Journal, p. 70). This bulletin gives an analytical procedure for the structural analysis of wingwalls of variable thickness. As an example, a typical variable thickness wingwall is analyzed for moments which are presented as moment contours.

Deflections of an abutment wingwall considered as a thin trapezoidal plate

H. J. Weiss, *Bulletin* No. 185, Iowa Engineering Experiment Station, May 1959, 12 pp., $0.50

This bulletin presents a more detailed and rigorous analytical and numerical solution to the problem of a constant thickness wingwall. This study was undertaken to determine whether certain simplifying assumptions which were used in the previous bulletins (see review above), and which considerably reduced the complexity of the problem, are justified insofar as the desired accuracy of results obtained are concerned. The conclusion is that for engineering purposes this more detailed solution is not worth the extra time and expense. It does show, however, how the analytically difficult problem of plates with nonrectangular corners may be handled.

Construction

Heliport in the center of London

Engineering (London), V. 187, No. 4859, Apr. 24, 1959, pp. 542-543

Reviewed by Aron L. Mirsky

Brief report on completion of one of the first of a new genus of facilities. Heliport, on Thames, is T-shaped, with a 125 x 53 ft landing and takeoff pad and a 50 x 65 ft stem between pad and shore. Structurally, facility has a 21 in. reinforced concrete slab on prestressed concrete or hollow steel box piles; design was based on a helicopter weighing 15 tons, landing on its two main wheels simultaneously, with an impact velocity of 10 fps and a dynamic load factor of 2.

Reviewer hopes technical paper with full details will achieve early publication.

Curtain wall construction

Building Construction Illustrated, V. 27, No. 5, May 1959, pp. 38-105

Summary of information available on curtain wall construction. Outstanding examples of unit masonry in modern curtain walls plus

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characteristics, specifications, and construction detailing. Details and characteristics of all types of thin panels currently in use—what they are, what they do, how they are applied, and who makes them. Guide to resilient type joints, seals, and mastic type sealers, and details of their placement—includes characteristics, life expectancy, and general conditions of use.

Some observations on buildings with concrete walls cast in place (in Rumanian)

S. Fleser, *Industria Constructiilor si a Materialeor de Constructii* (Bucharest), V. 10, No. 12, 1958, pp. 613-616

Reviewed by J. J. Polivka

Properties and developments of monolithic concrete walls used in the construction of houses, and advantages and disadvantages, are discussed. Also reports on the costs of the construction. Comparison is made with similar buildings in Germany, USSR, Austria, England, and United States.

Movable form elements (in Danish)

Axel Efsen, *Nordisk Betong* (Stockholm), V. 3, No. 2, 1959, pp. 105-117

Reviewed by Margaret Corbin

Article describes the construction of a hangar executed as a shell structure comprising three arches which in part were situated above the shell. The 37 x 30 m hangar was divided by the arches into four transverse bays, each 7.5 m long. Construction was carried out with the aid of a movable form section just large enough to cover one of these bays—37 x 7.5 m, which was re-used four times. It consisted of a large and deep center element and four smaller side elements of lesser depth. A cost estimate is included.

Concrete arches support Pittsburgh apartment building

R. M. Gensert and Emil C. Hach, *Civil Engineering*, V. 29, No. 6, June 1959, pp. 47-49

The Neville Street Apartment Building, Pittsburgh, consists of 11 stories, including basement. The entire structural system is of reinforced concrete. The ground floor is open, with concrete arches at this level serving as a colonnade leading to the centrally located elevator shaft. Additional support for the building comes from concrete walls at elevator and stair towers located in the central part of the building.

In the appearance of this building, the structural form was subordinated to the clever manipulation of shape and color, yet the structural frame promotes function and use.

Discusses the architectural plan and the model analysis to evaluate the structural behavior. The Begg's Deformeter method was used on an elastic material model. Table gives comparison of values for shear, thrust, and moment under two design assumptions and the model analysis.

Construction Techniques

Mechanization in housing construction (in Slovene)

L. Treppo, *Gradbeni Vestnik* (Ljubljana), V. 9, Special Issue, 1958, pp. 67-71

Reviewed by J. J. Polivka

Presents comparative analyses on concrete buildings and others to show that construction costs can be reduced by 5-9 percent (savings in manpower about 25 percent) by using proper mechanization. Comparatively small concrete mixing plants and tower cranes, assembled on the site in 5-10 hr, proved economical.

Criteria of economical realization of shell roofs in Poland (in Polish)

K. Thiel, *Inzynieria i Budownictwo* (Warsaw), No. 1, 1958, pp. 1-6

Polish Technical Abstracts No. 3 (31), 1958

Discusses two methods of constructing shell roofs. One employs traveling forms and the other uses prefabricated elements. Laboratory experiments and field tests are given in detail.

Cost of placing stiff concrete

Concrete and Constructional Engineering (London), V. 54, No. 3, Mar. 1959, pp. 125-127

Presents a study to determine if the savings made due to reduced use of cement in stiff concrete is nullified or exceeded by the additional cost entailed in placing this concrete.

Three examples are given, each using a different volume of concrete and at different job sites. Results indicate that where the

rate of placing the concrete is slowed down because of using a dry mix the effect on the total cost is to increase it. This is brought about by the need for additional labor required to spread and compact the stiff concrete.

Three columns cast in sliding forms (in Swedish)

SVEN THAULOW, *Nordisk Betong* (Stockholm), V. 3, No. 1, 1959, pp. 77-83
Reviewed by MARGARET CORBIN

Three 70 ft high concrete columns erected at the Building Exhibit in Oslo in 1958 were cast in less than 29 hr with the use of 4 ft long sliding steel forms. The high rate of concreting of 32 in. per hr was made possible through: (1) use of high-early-strength cement, (2) heating the mixing water to 140 F, and (3) use of a 2.5 percent calcium chloride admixture related to cement content. Cylinder strength was tested after 4 and 8 hr as well as at 7 days. In this case the rate of concreting was greatly accelerated to demonstrate utmost limits of performance. Precautions based on a careful evaluation of conditions are recommended.

Dams

Production and handling of concrete materials—Tinaroo Falls Dam

A. J. HILLESS, *Journal*, Institution of Engineers, Australia (Sydney), V. 31, No. 3, Mar. 1959, pp. 69-80
AUTHOR'S SUMMARY

This paper discusses the design and selection of a plant which was required for the production and handling of aggregates for a major dam project. A description of the plant installed at Tinaroo Falls Dam is followed by an analysis of its efficiency. As a result of the experience gained at Tinaroo Falls, the author indicates steps which should be taken to obtain a higher efficiency for a plant of similar capacity.

Shell-type dams

MAX HERZOG, Translation from *Die Bautechnik* (Germany), V. 33, No. 2, 1956, pp. 47-54; 1958, 24 pp. (Order from SLA Translation Center, John Crerar Library, Chicago: Translation No. 59-10746; microfilm, $2.70; photocopy, $4.80)
TECHNICAL TRANSLATIONS
V. 1, No. 11, June 1959

An analysis of shell dams is presented with design computations based on the theory of circular cylindrical shells. It is shown that the bearing capacity of a shell dam is a result of its combined action as a membrane and as a slab. By arbitrarily separating these two effects in computations, a convenient approximation method is developed for design. It is shown how the assumption of allowable stresses is affected by the use of the theory of probability and of a statistical evaluation of test results obtained on materials. The problem of stability, which increases in importance when reduction of dam thickness is made possible, is briefly discussed. Proof is given that shell-type dam structures are completely safe.

Design

Elastic resistance of reinforced concrete beams

G. R. SWIHART, J. R. ALLGOOD, and W. A. SHAW, *Proceedings*, ASCE, V. 85, No. ST1, Jan. 1959, pp. 43-63

A theory is presented for determining the static resistance characteristics of uniformly loaded reinforced concrete beams. Particular attention is given to predicting ultimate deflection. Four loading conditions are considered. The theory is compared with static test data.

Structures to withstand earthquakes

D. A. HOWELLS, *Engineering* (London), V. 187, No. 4853, Mar. 13, 1959, pp. 346-349
Reviewed by ARON L. MIRSKY

A brief review of the characteristics of earthquakes, the response spectrum, their idealization as a random series of impulses, the response of complex structures to dynamic loads, shear distribution in tall buildings, and nonlinear behavior, leads to the main portion of paper, a discussion of building codes and good design practice in earthquake territory.

Reinforced concrete columns subjected to bending (in Swedish)

ASGER G. T. WAGEPETERSEN, *Nordisk Betong* (Stockholm), V. 3, No. 1, 1959, pp. 47-65
Reviewed by MARGARET CORBIN

Presents a method for calculating the maximum load-carrying capacity of reinforced concrete columns subjected to bending. The method is based on the assumption that

the cross-sectional area of the reinforcement is replaced by an equivalent expression in terms of n times the cross-sectional area of concrete. The ability of the concrete in tension to transmit tensile stresses is disregarded in this connection. It is assumed that the stresses increase in proportion to the distance from the neutral axis. A numerical example is given.

Understanding the hyperbolic paraboloid

FELIX CANDELA, *Architectural Record*, No. 7, July 1958, pp. 191-195; No. 8, Aug. 1958, pp. 205-207, 215

"Of all the shapes we can give to a shell, the easiest and most practical to build is the hyperbolic paraboloid." With this as his thesis, Candela decrys the use of arbitrary shapes for structures and examines the logic of the hyperbolic paraboloid in terms of "proper" structural behavior, simplicity of stress analysis, and ease of construction. Part 1 summarizes basic shell action, types of shells, and defines the geometry of the hyperbolic paraboloid. Part 2 presents stress analyses for various types of hyperbolic paraboloids.

Linear structural analysis

P. B. MORICE, The Ronald Press Co., New York, 1959, 170 pp., $6

An introduction to the influence coefficient method applied to statically indeterminate structures. Subject is that of the equilibrium analysis of linear elastic structures using the exact flexibility method of structural analysis in which equilibrium states are combined to give geometrical compatability. Applies the strain energy method to skeletal or one-dimensional frame structures. Discusses strain energy and indeterminacy and develops influence coefficient equation. Defines matrices and discusses various types and methods of handling, then applies the principles of matrix algebra to computations and gives numerical examples for solutions of symmetrical and unsymmetrical matrices. In sections dealing with structures discusses the stress resultant distribution due to initial lack of fit of structural members and to settlement of supports. Describes release system, relations, and effects on flexibility matrix. Discusses applications of electronic digital computers to solutions of this method of analysis. Appendix contains four further examples of problems of structural analysis in which the outline of the working is indicated.

Flat slabs supported on column. II (in Swedish)

ROLF BAEHRE, *Nordisk Betong* (Stockholm), V. 3, No. 1, 1959, pp. 15-21

Reviewed by MARGARET CORBIN

The present investigation gives a qualitative idea of the structural action of a flat slab carried on a column in an interior panel. The results, which were obtained at two extreme values of the span ratio, permit certain conclusions which are of general importance in the design of flat slabs supported along all edges and carried on a column in an interior panel. It is possible in principle to generalize these results so that they may be applicable to flat slabs which are wholly or partly restrained along the edges.

Improved iteration method for continuous beams (Verbessertes Iterationsverfahren für durchlaufende Balken)

H. CRAEMER, *Der Bauingenieur* (Berlin), V. 33, No. 9, Sept. 1958, pp. 336-338

Reviewed by ARON L. MIRSKY

The standard three-moment equation is solved for the middle unknown in terms of the loading and the other two unknowns, and a numerical solution obtained by a rapidly-converging iteration. Errors are said to be self-correcting, unlike moment distribution. Worked examples include various types of loading, end conditions, influence lines, and support settlement.

The method is extended by Moser in title source, V. 34, No. 1, Jan. 1959, pp. 38-39.

Notes on the design of structures founded on heaving soil

C. F. TEMPLER, Symposium on Expansive Clays, Part II, *Transactions*, South African Institution of Civil Engineers (Johannesburg), V. 7, No. 12, Dec. 1957, pp. 387-392

Reviewed by D. G. NORMAN

Philosophy of design outlined briefly and selection of type of treatment. Acceptable degrees of differential settlement are reviewed. Considers use of (1) lime mortar to allow slight movement of bricks, (2) sealing apron around building, (3) soaking pits to keep soil

saturated, (4) reinforced strip footings (not found effective), (5) reinforced brickwork (successful), (6) reinforced concrete string courses in brickwork, (7) reinforced foundation beams, (8) raft foundations, (9) three-point support, (10) full framing, (11) removal of expansive material, (12) structural steel framing, (13) division into units, (14) provision for jacking, and (15) under-reamed piles.

On ribbed decks supported by torsionally-rigid edge girders (Über die auf torsionssteife Randträger aufliegende Rippendecke)

ADNAN ÇAKIROGLU, *Der Bauingenieur* (Berlin), V. 33, No. 8, Aug. 1958, pp. 294-299
Reviewed by ARON L. MIRSKY

Develops a governing differential equation and solutions for decks on torsionally rigid, infinitely long edge beams with spans of equal length. Also discusses solution for edge beams of finite length or unequal span lengths. Two types of loading are considered—normal to deck, and parallel to deck. Method of solution is analogous to the Hardy Cross method, with certain simplifying assumptions. Two charts are given to aid in the solution of the case involving normal loading along with three worked examples.

Overload factors can cause ultra-conservative design

RICHARD N. BERGSTROM, *Proceedings*, ASCE, V. 85, No. ST2, Feb. 1959, pp. 71-79

This paper reviews the use of overload factors in establishing design stresses for reinforced concrete and prestressed concrete structures and for steel transmission towers. It compares this with results of conventional design stress assumptions and finds overload factors create a much higher safety factor than would normally be expected.

Design of prismatic structures

A. J. ASHDOWN, Concrete Publications Ltd., London, Revised Edition, 1958, 85 pp., $2.10

Revised edition gives examples of the design of prismatic slab roofs and other structures such as trough bunkers which comprise an assembly of contiguous and continuous rectilinear slabs which are also known as "hip plate structures." Minor alterations in the book bring the material up to date and new information is given on the analysis of prismatic slab roofs, in which the angle between adjacent slabs is much less than 30 deg—that is, slab roofs and profiles of which approximate a curve. A design for a prestressed concrete edge beam for such a roof is also given.

Aseismic design of structures by rigidity criterion

EDWARD Y. W. TSUI, *Proceedings*, ASCE, V. 85, No. ST2, Feb. 1959, pp. 81-106

It has become apparent that the earthquake design coefficients of many building codes are insufficient for certain types of structures, particularly for industrial and other, more specialized types. A set of curves is presented here which show design coefficients as function of the rigidity of structures. These coefficients are based on the modified response spectra of one-mass systems obtained by dynamic analysis.

It is hoped that the paper may serve as supplementary reference for the conventional aseismic design of special structures and that it will evoke discussion which may lead, ultimately, to the establishment of a generalized seismic design criterion.

Static work of conoidal shells in the light of experiments (in Polish)

B. KOT, *Inzynieria i Budownictwo* (Warsaw), No. 2, 1958, pp. 37-45
POLISH TECHNICAL ABSTRACTS No. 3 (31), 1958

Presents a method for an analytical determination of the stresses in a conventional shaped conoidal shell. Membrane forces, stability of the conoid, and forces acting on the upper and lower arch are determined.

Analysis of frost action beneath cold storage warehouses

D. C. PEARCE, *Transactions*, Engineering Institute of Canada (Montreal), V. 2, No. 4, Dec. 1958, pp. 153-156
AUTHOR'S SUMMARY

The temperature distribution beneath a cold storage warehouse has been determined approximately by analytical methods. This analysis has demonstrated that freezing of the soil to a considerable depth beneath a warehouse of slab-on-grade construction is to be expected. Thus, there exists the possibility of damage by frost action to such a

building. The elimination of this possibility of damage by the introduction of heat beneath the floor insulation is shown to be economically feasible. The design of a heating system to prevent soil freezing is discussed in detail.

Materials

Reaction of water on portland cement (La réaction du ciment portland avec l' eau)

A. R. Steinherz and N. Welcman, *Revue des Matériaux* (Paris), No. 517, Oct. 1958, pp. 265-271
Reviewed by Phillip L. Melville

The complex reactions of portland cement and water have been studied by chemical analysis of the solution formed between the hydrating grains of clinker. The liquid is characterized by a pH of 12.5. Its chemical composition is not influenced by the fineness of the cement (hydration may be a solid state process). The gelatinous layers around the grains absorb part of the alkalies.

Some laboratory test results on fly ash and pozzolanic cements (Quelques resultats d' experiences de laboratoire sur less cendres volantes et les ciments aux cendres)

A. Jarrige and R. Ducreux, *Revue des Matériaux* (Paris), No. 518, Nov. 1958, pp. 300-305
Reviewed by Phillip L. Melville

In connection with the preparation of new standards, test methods were studied to determine the amount of fly ash and pozzolon active in cements. Two methods are recommended for rapid checks: solubility in HCl and percentage of insoluble matters after burning for 1 hr at 1000 C.

Properties of road materials and progress in their location and testing

D. F. Orchard, *Journal*, Institution of Engineers, Australia (Sydney), V. 31, No. 4-5, Apr.-May, 1959, pp. 118-224

No branch of engineering or of scientific endeavor can make real progress until accurate methods of testing the materials used and of measuring the results obtained can be devised. These are the prerequisites of replacing empirical methods with those of scientific accuracy.

Road construction has perhaps remained in the empirical realm longer than any other branch of civil engineering, but the testing of road materials is rapidly becoming a highly specialized subject involving the services of physicists and electronic engineers, as well as those of the civil engineer. This article discusses the problem of testing the finished road and also the location of road construction materials. Describes tests and properties of the aggregate, the concrete, or the bituminous mixtures. The section on nondestructive methods of test is interesting and includes a table of four classifications of nondestructive methods of testing concrete. This tabulates type of test, testpiece, parameters, applications, advantages, and disadvantages.

Precast Concrete

New potential: Precast paving stones

Concrete Products, V. 62, No. 4, Apr. 1959, pp. 36-38, 55

Describes the potential use of precast paving stones in the United States for traffic lanes, driveway markers, cross walks, and parking stalls.

Wearability and placement limit these stones from being used for complete paving of streets but other uses, such as those mentioned above and for shopping centers and factories, may be more applicable.

Complete story of colored block

Richard H. Green, *Concrete Products*, V. 62, No. 2, Feb. 1959, pp. 34-41

Concrete masonry products have been successfully offered in color throughout the Southwest and Florida. A suggested criteria for selecting the most suitable pigments is given. It is stated that these are either natural or synthetic mineral pigments.

Prestressed Concrete

Developmental status of prestressed concrete (Entwicklungsstand des Spannbetons)

Wolfgang Zerna, *VDI Zeitschrift* (Düsseldorf), V. 101, No. 5, Feb. 11, 1959, pp. 157-163
Reviewed by Aron L. Mirsky

Paper, one of four* on steel and concrete structures at the VDI General Meeting, Cologne, June 11, 1958, affords a good glimpse

*The two papers which pertain strictly to steel will not be reviewed here. Suffice it to say that both are highly interesting and that the perusal of both is highly recommended to concrete engineers, as good for the soul!

of the current status of the art, first in the way of developments in materials and their applications, and then in the form of structures both completed and contemplated. Included in the last is a print of "Tomorrow's Bridge" which appeared in a Universal Atlas Cement Co. advertisement in 1957, and which has been arousing interest and comment in this country ever since. Paper is recommended reading despite a few misstatements, such as placing Lake Pontchartrain in California.

Roofing large areas with reinforced and prestressed concrete (Überdecken grosser Räume mit Stahl- und Spannbeton)

GOTTHARD FRANZ, *VDI Zeitschrift* (Düsseldorf), V. 101, No. 5, Feb. 11, 1959, pp. 180-188
Reviewed by ARON L. MIRSKY

Paper, another of the four on steel and concrete structures at the VDI General Meeting, Cologne, June 11, 1958, is especially noteworthy in that major emphasis is placed on structural *form* to meet not only the usual needs of loading, heating, access, acoustics, light, economy, etc., but to preserve the feeling of space. A fascinating array of solutions is described, ranging from the by now fairly common rigid frames to singly and doubly curved shells and folded plates.

Properties of Concrete

New contributions to the study of cracking of concrete (Nouvelles contributions a l'étude de la fissuration du béton)

Supplement to *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), V. 11, No. 132 (Series 51), Dec. 1958
AUTHOR'S SUMMARY

Shrinkage and cracking of coatings (Retrait et fissuration des enduits)

M. C. LEFEVRE, pp. 1269-1277

A method is proposed which makes it possible to observe the behavior of a coating under the influence of the principal parameters affecting shrinkage and tendency to cracking. Measurements of shrinkage and of cracking time are made after application of the coating.

Results obtained show the importance of shrinkages which may take place in the first few hours after mixing. The tests furnish data which will be of value in determining proportions of cement, water, fine aggregates, use of curing compounds, and curing during the first few hours. The test method makes it possible to determine rapidly the efficiency of actual methods on the work site.

Shrinkage and cracking of remixed mortars (Retrait et fissuration des mortiers rebattus)

CHAMBRE SYNDICALE DES CONSTRUCTEURS EN CIMENT ARME, pp. 1278-1286

This short study gives the results of measurements of shrinkage and of tests made to determine the tendency to cracking of neat paste and mortars for two different cements, as a function of the remixing time.

Test method for cracking of concrete (Methode d'essai de fissuration du beton)

M. C. LEFEVRE, pp. 1287-1290

The test method makes it possible to obtain cracks in concrete under the effect of shrinkage in periods varying from 2 to 3 weeks, depending on the composition of the concrete and on curing conditions. Several tests were performed with various concrete mixes (type of cement, proportioning, aggregate grading, addition of plasticizers) and under different curing conditions.

Concrete hardening at low temperatures (below zero) (in Polish)

S. KAUFMAN, W. KROL, E. LEBDA, and T. WOJTAN, *Materialy Budowlane* (Warsaw), No. 11, 1957, pp. 321-330
POLISH TECHNICAL ABSTRACTS No. 3 (31), 1958

Reviews the effect of artificially frozen ground on freshly placed concrete. Experiments and tests were conducted to determine the effects of sub-freezing temperatures on concrete made from several different cement types. Alumina cement gave the best results.

Excessive shrinkage of aggregates as a cause of deterioration of concrete structures in South Africa

N. STUTTERHEIM, Reprint from *Transactions*, South African Institution of Civil Engineers, V. 4, No. 12, Dec. 1954, and V. 5, No. 6, June 1955, 20 pp. (discussion included). Available from National Building Research Institute, South African Council for Scientific and Industrial Research, Pretoria.
AUTHOR'S SUMMARY

Peculiar failures have been observed in concrete structures at Graaff Reinet; the main features are large deflections in beams, floors,

and cantilevers; cracking of concrete coincident with reinforcement; corrosion of reinforcement; and severe map cracking. The cause has been traced to the use of aggregates which impart to the concrete a tendency to shrink excessively. Shrinkage of this order has not previously been reported from elsewhere in the world.

The coarse aggregates are fine-grained sandstones and the fine aggregates, river sands composed largely of particles of shale and sandstone, both materials being derived from the Beaufort series, and, it is suspected, also from adjacent formations of the Karroo system.

Tests to distinguish such materials from normal aggregates are discussed, and recommendations made with regard to existing and new structures in the areas affected.

Structural Research

Tests on concrete tunnel linings

CH. SZECHY, *Concrete and Constructional Engineering* (London), V. 54, No. 3, Mar. 1959, pp. 105-109

A study of segmented reinforced concrete tunnel linings. The design requirements are: (1) ability to support loads with smallest possible weight, (2) watertightness of joints and segments against high pressure heads, (3) sufficient strength of surface to resist thrust of jacks propeling the tunneling shield, (4) resistance to corrosion and chemical attack, (5) weight and dimensions of segments should be such to ensure ease of handling during delivery and erection.

Descriptions of the various tests are given, such as longitudinal and transverse loading tests, watertight tests, and chemical tests.

Investigations of stresses in shells using the photoelastic frozen-stress method (Spannungsuntersuchungen an Schalen nach dem spannungsoptischen Einfrierverfahren)

THEODOR GAYMANN, *Forschung auf dem Gebiete des Ingenieurwesens*, VDI (Düsseldorf), V. 25, Forschungsheft 471, 1959, 36 pp.

Reviewed by ARON L. MIRSKY

Three cases of shells (more accurately, pipe sections) were intensively studied by the frozen-stress technique, using Araldit B. The three main sections of this paper, based on author's dissertation (Technische Hochschule München, 1957), cover: (1) the elastic and photoelastic properties of Araldit B (pp. 3-10); (2) errors, due mainly to nonsimilarity of model and prototype, which restrict the types of shells which can be studied effectively by this method (pp. 11-14); and (3) experiments (procedures, methods of evaluation, results) on three pipe sections: a circular cylindrical shell (pipe) uniformly loaded radially; a circular cylindrical shell (pipe) with two equal, opposite concentrated radial loads; and a pipe bifurcation (Tee) under internal pressure.

Energy versus compatibility analogs in electrical simulators of structures

FREDERICK L. RYDER, *Journal of the Aero/Space Sciences*, V. 26, No. 2, Feb. 1959, pp. 108-116

Reviewed by ARON L. MIRSKY

Usual simulator is based on simulation of force and moment equilibrium relationships by current-continuity relationships and on simulation of compatibility of deformations by compatibility of voltages; considerable theoretical analysis is usually required to identify the voltages and their analogous deformations. In second type of simulator, energy in the structure is simulated by resistive power loss (static case) or inductive and capacitive energy (dynamic case); simulator automatically computes and satisfies the compatibility requirements. Not only is required theoretical treatment much simpler, but resulting circuit requires fewer components and is less sensitive to component imperfections than in first type of simulator; includes discussion of method for several interesting structural forms.

Execution of skewed continuous slab bridges (Ausführung schiefwinkliger kontinuierlicher Plattenbrücken)

L. SCHMERBER, G. BRANDES and H. SCHAMBECK, *Der Bauingenieur* (Berlin), V. 33, No. 5, May 1958, pp. 174-183

Reviewed by ARON L. MIRSKY

Results of mathematical analyses and of experimental studies (using "Presan"-type methods) are compared, and details of the actual construction and reinforcement are given for the Kötzting Regenbrücke, a two-span structure. Investigated idealized models both with and without haunches (the skews of the abutments and piers were all different in the prototype).

ACI News Letter

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September 1959

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12th regional meeting

Mexico City

November 2 to 5

With the lure of fascinating, cosmopolitan Mexico City as a background plus the usual top-notch ACI technical and social programs, the 12th regional meeting scheduled for November 2-5 promises to be a highlight in Institute history. Facilities of the Hotel Continental Hilton and the Hotel Del Prado will accommodate Institute members attending the meeting, with various meetings and social functions being conducted at the hotels and at University City.

Registration opens Monday, November 2 at 10 a.m. in the main lobby of the Hotel Continental Hilton.

Our genial hosts South of the Border will welcome ACI'ers to the meeting on Monday evening at a "Kick-off" party at the Continental Hilton. The 3-day technical, bilingual program will feature outstanding concrete authorities from Cuba, Mexico, and the United States. About 20 papers on concrete problems and construction techniques of prime importance in Mexico and southwestern United States will be presented at five technical sessions devoted to quality of concrete; reinforcement; concrete structures; concrete shells; and precasting. The technical sessions will be conducted at University City, known as the "Campus in the Clouds." Here the heritage of ancient civilization finds a glorious complement and contrast in the many magnificent buildings of functional architecture designed by Mexico's most

Exclusively for the ladies

Senora Marcela Castro Villagranada de Hernandez has arranged a special ladies program to help make their trip to Mexico City a truly memorable holiday.

Agenda calls for sightseeing tours, a fashion show, luncheon at the Ciudad Universitaria and at Xochimilco, friendly coffee get-to-gethers and guided shopping sprees.

famous architects and decorated in oils and tile by Diego Rivera murals. A luncheon is scheduled at University City on Tuesday, November 3. Technical sessions will convene at 9 a.m. and 4 p.m., November 3-5, allowing ample time for leisurely lunching and midday sightseeing.

On Wednesday, November 4, the technical program will be interrupted with lunch in Xochimilco, the City of Floating Gardens. Only 10 min. away from modern Mexico City, Xochimilco is a scenic wonder, with canopied barges decked with flowers and colorfully costumed Indian boatmen gliding through the beautiful tree-bowered canals.

On Thursday evening, November 5, the local committee has planned a dinner-dance and reception at the Hotel Del Prado.

For those who have time to linger in Mexico City after the close of the regional meeting, the local committee has tentatively arranged a number of tours ranging from 1 to 5 days to help you enjoy Mexico to the fullest. In addition to a "get-acquainted" tour of Mexico City to view the National Palace, Chapultepec Castle, the Street of Silversmiths, the great cathedrals, San Juan Market and other wonders of the city, the committee suggests trips to include such scenic sites as the Pyramids of the Sun and Moon, ancient altars of worship of the Toltec and Aztec tribes, where you can view the fantastically beautiful carvings of the ancient Indians; or in direct contrast, a trip to Acapulco, the exotic spot that has tropical gardens, deep sea fishing, picturesque sights and gay social life.

The committee also plans to arrange tours for stopovers at Puebla, a magnificient old city, famous for its pottery and many buildings resplendent with Moorish touches of majolica domes and mosaic tiles; also the city of Guanajuato, located in a canyon with many narrow, winding cobblestone streets, once the site of some of the richest silver veins of Mexico. Also recommended as a "must" on your itinerary is sparkling Guadalajara, Mexico's second largest city, sometimes referred to as "The Pearl of the West;" and picturesque, Spanish-Mayan Merida, capital of the state of Yucatan.

Alberto Dovali Jaime, Universidad Nacional de Mexico, is general chairman of the regional meeting with Adolfo Zeevaert, Torre Latino Americana, serving as secretary, and Antonio Robles, Representante de Cemento Mixcoac, acting as treasurer. Leonardo Zeevaert, consulting engineer, heads up the planning group for the technical program which appears below.

TECHNICAL PROGRAM

TUESDAY MORNING, NOV. 3

Stress Analysis in Hyperbolic Paraboloids Considered as Translation Surfaces
Ignacio Canals and Jorge Molina Montes

Analysis and Design of Warped Surfaces
Ignacio Canals and Guillermo Gargollo

Special Conference

TUESDAY AFTERNOON, NOV. 3

Prestressed Concrete Bridge Construction in Mexico
Vicente Guerrero y Gama

Elastic Analysis of Shear Walls in Tall Buildings
Emilio Rosenblueth and Ignacio Holtz

Prestressed Precast Concrete Arches Span 102 ft 6 in. for Industrial Roof
E. R. Cancio and Angel Herrera

A Multiple Purpose Concrete Building Built of Precast Thin Shell Panels
A. Amirikian

WEDNESDAY MORNING, NOV. 4

Reinforced Concrete Hyperbolic Paraboloidal Shells
Howard P. Harrenstien

Folded Plate Design for Reinforced Concrete Raft Foundation of a 24 Story Building
Ignacio Martin and Sixto Ruiz

Behavior and Strength in Shear of Reinforced Concrete Beams and Frames Without Web Reinforcement
R. Diaz de Cossio and C. P. Siess

General Formulas for Membrane Stresses in Hyperbolic Paraboloid Shells
Felix Candela

Special Conference

WEDNESDAY AFTERNOON, NOV. 4

Reinforcing Steel
Luis Gimenez Cacho

Special Steels for Concrete
Compania Fundidora de Fierro y Acero de Monterrey, S.A.

Cold Twisted Steel Produced in Mexico
Max Calva Caudrilla

THURSDAY MORNING, NOV. 5

Concrete Dams
W. Simmons (Presented by Fredrico Barona de la O.)

Ready Mixed Concrete
Livio Pontecorvo

A Rigid Prestressed Concrete Tie for Mexico
Alberto Dovali Jaime

Four Decades of Improved Concrete Technology for Water Resource Development
Grant Bloodgood

Special Conference

THURSDAY AFTERNOON, NOV. 5

Special Conference

General Comments
Leonardo Zeevaert

Official Closure

Special conferences will be given by some of the following international concrete authorities: C. P. Siess, United States; E. Torroja, Spain; U. Finsterwalder and H. Meier, Germany; Y. Guyon, France; and Felix Candela, Mexico.

PROGRAMA TECHNICO

MARTES A. M., NOVIEMBRE 3

Analisis de Esfuerzos en el Paraboloide Hiperbolico Considerado Como Superficie de Translacion
Ignacio Canals y Jorge Molina Montes

La Velaria, Su Analisis y Diseno
Ignacio Canals e Guillermo Gargollo

Conferencia Especial

MARTES T. M., NOVIEMBRE 3

La Construccion de Puentes de Concreto Precomprimido en Mexico
Vicente Guerrero y Gama

Analisis Elastico de Muros Rigidos en Edificios Altos
Emilio Rosenblueth e Ignacio Holtz

Arcos de Concreto Precolados y Preesforzados, de 31.24 m. de Claro, Para un Techo Industrial
E. R. Cancio e Angel Herrera

Un Edificio de Concreto Para Usos Multiples Construido de Cascarones en Tableros Precolados
Arsham Amirikian

MIERCOLES A. M., NOVIEMBRE 4

Cascarones Paraboloide Hiperbolicos, Tipo Sombrilla, de Concreto Reforzado
Howard P. Harrenstien

Diseno de Una Trabe-Losa Corrida de Concreto, Para Cimentacion de un Edificio de 24 Pisos
Ignacio Martin e Sixto Ruiz

Comportamiento y Resistencia al Esfuerzo Cortan te de Vigas y Marcos de Concreto Sin Refuerzo en el Alma
Roger Diaz de Cossio y Chester P. Siess

Formulas Generales Para Los Esfuerzos de Membrana en Cascarones Paraboloide Hiperbolicos
Felix Candela

Conferencia Especial

MIERCOLES T. M., NOVIEMBRE 4

Aceros de Refuerzo
Luis Gimenez Cacho

Aceros Especiales de Refuerzo Para Concreto
Compania Fundidora de Fierro y Acero de Monterrey, S. A.

El Acero Torcido en Frio Que se Produce en Mexico
Max Calva Caudrilla

JUEVES A. M., NOVIEMBRE 5

Presas de Concreto
Warren Simmons (Presentacion del Ing. Federico Barona de la O.)

Concreto Premezclado
Livio Pontecorvo

Un Durmiente de Concreto Precomprimido, Tipo Rigido, Para Mexico
Alberto Dovali Jaime

Cuatro Decades de Tecnologia Majorada de Concreto Para el Desarrollo del approvechamiento de Recursos Hidraulicos
Grant Bloodgood

Conferencia Especial

JUEVES T. M., NOVIEMBRE 5

Conferencia Especial

Comentarios Generales Sobre la Reunion
Leonardo Zeevaert

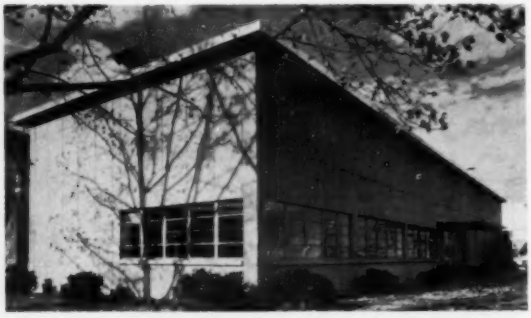
Clausura Oficial de la Doceava Reunion Regional

Las conferencias especiales estaran a cargo de algunas de las siguientes personas: Prof. Dr. Chester P. Siess, Estados Unidos; Prof. Dr. Eduardo Torroja, Espana; Dr. Ing. Herrmann Meier, Alemania; Dr. Ing. Ulrich Finsterwalder, Alemania; Ing. Dipl. Yves Guyon, Francia; Arqto. Felix Candela, Mexico.

Felix Candela, architect, has arranged a number of inspection trips to further augment the technical program.

Much of the credit for the plans of the local committee is credited to Luis Flores Arias, civil engineer, in charge of finance; Rafael Hernandez Hernandez, civil engineer, overseeing social events; and Enrique Torres de Alba, La Consolidada, S. A., handling publicity.

Balance of the planning committee is comprised of Alfonso Marin, E., civil engineer, membership promotion; A. Pardo Boland, consulting engineer, exhibits and displays; and Francisco Santos Oliva, civil engineer, educational contacts.



PRECAST and prestressed concrete were used throughout the new office and change house building constructed at the Universal, Pa., plant of Universal Atlas Cement, a division of United States Steel Corp.

Large precast reinforced concrete sections 6 in. thick, 19 ft long, and 2 to 7 ft high, form the exterior walls and are attached to the column faces by means of a cast-in-place reinforced key. A uniform panel effect is provided by precast vertical grooves or reveals. The exterior walls are striated and painted with two coats of gray cement base paint.

The frame of the 40 x 140-ft building is constructed of precast reinforced concrete columns and post-tensioned beams supporting a precast second floor. Post-tensioned concrete members, approximately 40 ft long, support the precast roof sections. The ground floor was a monolithic slab.

The building was designed by Herbert Fleischer, New York, and constructed by Rust Engineering Co., Pittsburgh.

275 miles of wire to prestress concrete reactor shield

THE UPPER SECTION of the cylindrical concrete radiation shield which surrounds the spherical containment vessel for Consolidated Edison's new reactor at Indian Point, N. Y., will be wrapped with some 275 miles of prestressing wire to enable it to withstand the horizontal forces exerted by the roof. The roof will be in the form of a dome and will be supported by the cylindrical wall. The top of the wall is beveled inward and has 24 pockets into which will fit precast concrete roof struts.

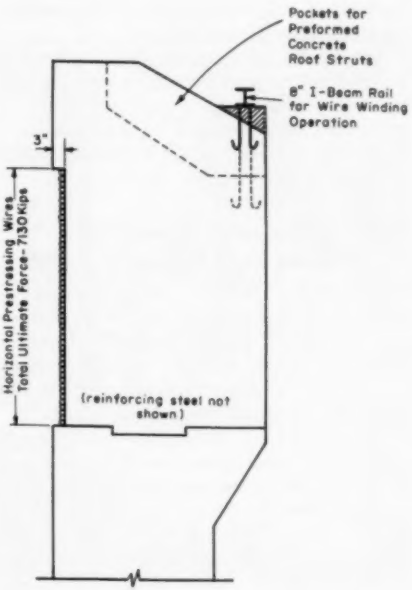
The shielding wall extends approximately 90 ft above grade and has a 181 ft outside diameter. The bottom 70 ft of the wall is 5 ft 6 in. thick. The top 20 ft or so widens inwardly to 7 ft 9 in. It is this widened section which is prestressed. A 3 in. deep chase has been cast in the outer surface to receive the prestressing wire. The chase is 4 ft 3 in. from the top of the wall and extends 15 ft 6 in. down the side of the cylinder.

The prestressing wire will be wrapped around the cylinder by a self-propelled machine suspended from a carriage that will travel around the top of the wall near the outer edge. The wire will be wrapped in six layers and will take 6 or 7 weeks to apply, roughly one layer per week. In all, there will be some 2530 wraps of wire, 163 wraps per ft. Pneumatic mortar flashing will be applied over each layer.

The initial wire size is 0.192 in. in diameter and this will be drawn to 0.174 in. as it is applied. This will impart a stress of 150,000 psi in the wire and will result in a total ultimate inward force on the structure of 7,130,000 lb. This force not only will counteract horizontal forces exerted by the roof, but will also withstand forces resulting from differences in inside and outside wall temperatures.

In prestressing a structure of this type, the carriage which supports the wire-winding machine is normally secured to cables terminating on the ground at the center of the cylinder. However, in this case, work on the containment vessel will be in progress during winding, so a special method of anchoring the carriage has been devised. A continuous 8-in. I-beam has been fastened to the top of the cylinder wall along the inner edge, and the carriage will be secured to a trolley which will travel along the I-beam as the self-propelled wrapping machine progresses. To install I-beam, anchor rods were cast in the wall and, since the beam is installed along a beveled surface, tapered blocks were fastened to these rods to provide a horizontal surface on which to secure the beam.

The prestressing operation will be performed by Preload Concrete Structures, Inc., Westbury, N. Y.


Pockets for Preformed Concrete Roof Struts
8" I-Beam Rail for Wire Winding Operation
3"
Horizontal Prestressing Wires
Total Ultimate Force - 7130 Kips
(reinforcing steel not shown)

Sketch of top of reactor shielding wall

Positions and Projects

Corbetta to head up local committee for '60 convention

Roger H. Corbetta, president of the Corbetta Construction Co., will chairman the local planning committee for the 56th annual convention of the American Concrete Institute to be held March 14-17 at the Commodore Hotel, New York.

The first 1½ days will be devoted to technical committee meetings with the first general session convening Tuesday afternoon, March 15. Several technical sessions will run concurrently Wednesday through Thursday noon followed by the final Research Session on Thursday afternoon.

The first session will feature proposed ACI Standards and committee reports.

Concurrent sessions on March 16 will be devoted to model tests, highways, materials, and design and structural research. On March 17 Committee 201, Durability of Concrete in Service, will present a symposium on restoration of deteriorated concrete. Committee 115 will conduct its annual research forum on Thursday afternoon.

The local planning committee is arranging entertainment and hospitality features for all conventioneers and several events are being arranged especially for the ladies.

Plan now to attend the 56th annual ACI convention in New York, March 14-17.

Zar appointed chairman of reactivated ACI Committee 505

ACI Committee 505, Design and Construction of Reinforced Concrete Chimneys, was reactivated as of June 1959. Changes in the operating methods of power plants necessitated a reappraisal of some of the provisions of the ACI 505-54 standard.

The Board of Direction recently appointed Max Zar, associate of Sargent and Lundy, Chicago, as chairman of the committee to restudy the present standard with a view of its suitability in the light of new developments and to recommend any modifications necessary to satisfy present or anticipated operating conditions in reinforced concrete chimneys.

H. M. Estes of Stone & Webster Engineering Corp., Boston; Russell A. Lahr, Rust Engineering Co., Pittsburgh; and Clem G. Merriman, Detroit Edison Co., Detroit, have been appointed to serve on the committee.

Other committee members include: E. W. Reeve of the M. W. Kellogg Co., New York; H. B. Schneider, Custodis Construction Co., Inc., Chicago; George Shervington, Consolidated Chimney Co., New York; and D. O. Thompson, Commonwealth Edison Co., Chicago.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 116, Nomenclature

William H. Wolf
U. S. Bureau of Reclamation
Denver, Colo.

Committee 216, Fireproofing or Fire Protection of Structures

E. W. Bauman
National Slag Association
Washington, D. C.

Committee 340, Ultimate Strength Design Handbook

Arthur J. Bunas
U. S. Bureau of Reclamation
Sacramento, Calif.

Alfred L. Parme
Portland Cement Association
Chicago, Ill.

Committee 624, Portland Cement Plaster

Arthur G. Eliel
Public Housing Administration
Washington, D. C.

Fred L. Knoblock
General Services Administration
Washington, D. C.

The U. S. Air Force Academy, Colorado
from the makers of . . .
Protex
PDA
FIELD PROVEN ECONOMY
in all phases of Concrete Placement!

PDA or Protex used on these projects

Among the many noteworthy and well-known projects around the world that are using PDA stands the name of the Air Force Academy in Colorado. Joining with this magnificent installation are dams, missile sites, highways, airfields, buildings . . . any construction where the efficient and economical placement of concrete is important!

PROTEX Dispersing Agent . . .

PDA increases strength at any age through water reduction and cement dispersion, improving all the desirable characteristics of concrete, giving a "live" concrete with protection against "hot" weather" slump loss and segregation — allows the placing of controlled durable air entrained concrete without plastic shrinkage cracks. PDA is a selective initial retarder— retarding only the initial set of concrete for extended vibration and finishing time (but delaying form stripping time) YET it gives no retardation in winter concreting — thus year around benefits are obtained with "all season" PDA.

Proven, Dependable, Adaptable

Proven in the field . . . PDA insures the successful and economical placement of better quality concrete!

Dependable — YES! Proof positive from Government and private projects. Also backed by the world-known and world-respected PROTEX name!

Adaptable to any need . . . PDA improves any Pre-stressed, Slip-form, Light-weight, Tunnel-lining, Tilt-up or Lift-slab concrete project!

PDA packaged in a durable bag comes to you in convenient powder form . . easily mixed and dispensed . . complete dispensing installations available upon request

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FOR BETTER AND MORE ECONOMICAL PLACEMENT OF CONCRETE, SPECIFY AND USE PDA . . . FROM THE MAKERS OF PROTEX!

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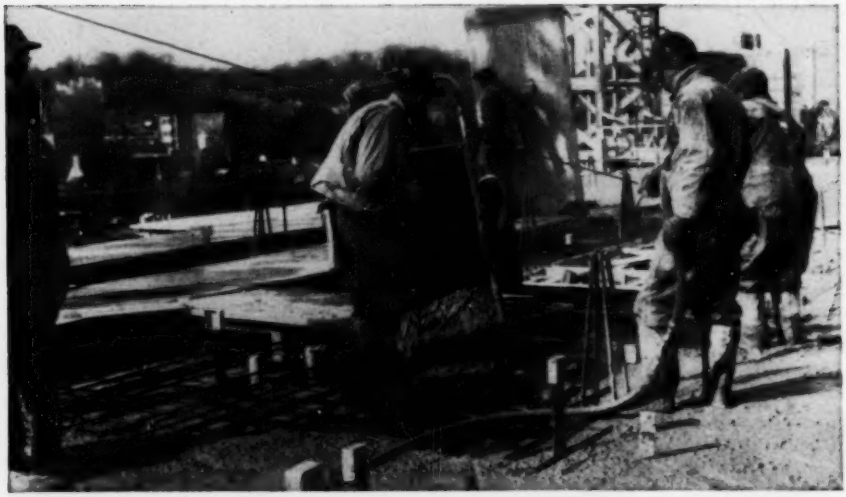

Concrete treated with Columbia Calcium Chloride sets faster. Cold weather protection time is shortened.

COLUMBIA CALCIUM CHLORIDE KEEPS COLD WEATHER CONCRETE JOBS *MOVING*

Don't let cold weather cost you money on your concrete jobs this fall. Keep your pouring schedules moving—and your profits up—by adding Columbia Calcium Chloride to your mix.

It's a matter of cost-record fact. At 70°F, concrete treated with Columbia Calcium Chloride reaches both initial and final set an average of *three times faster* than untreated mixes. With lower temperatures, the accelerating effect of Columbia Calcium Chloride is even more pronounced. When temperatures drop to 40°F, concrete treated with Columbia Calcium Chloride reaches three and seven day strengths in approximately half the normal time!

The result is that you meet strength specs days ahead, pull forms faster, get finishers on and off the job without profit-cutting overtime.

These specific advantages add up to the fact that *now* is the perfect time to analyze your batching procedures, or talk with your ready mix supplier. Make sure Columbia Calcium Chloride is a part of your next pour.

For more information on profit-making Columbia Calcium Chloride, just write to Calcium Chloride Department at our Pittsburgh address or to any of our fourteen conveniently located District Sales Offices.

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IN CANADA: Standard Chemical Limited

Erwin M. Lurie
Veteran's Administration
Washington, D. C.

Arthur T. Raitt
California Lathing and Plastering Contractors Association, Inc.
Los Angeles, Calif.

William A. Russell
Federal Housing Administration
Washington, D. C.

Committee 714, Concrete Bins and Silos

R. Neil Christy
Marietta Concrete Corp.
Marietta, Ohio

Committee 716, High Pressure Steam Curing

P. S. Bettoli
The Ruberoid Co.
South Bound Brook, N. J.

C. R. Hutchcroft
Keasbey and Mattison Co.
Ambler, Pa.

ACI building attracts visiting architects

The number of architects inspecting the ACI headquarters in Detroit continues to mount as the "Visitors' Register" reveals architects from Illinois, Indiana, Massachusetts, Michigan, Texas, and Buenos Aires, Argentina, among the many individuals viewing the building this month.

Permanente appoints Kendall to new sales management post

Edward H. Kendall has been appointed general sales manager of Permanente Cement Co. The new general sales management post was created to help Permanente match the continuing growth of its cement marketing activities. Mr. Kendall will headquarter in the Oakland, Calif., home office of Permanente.

Mr. Kendall joined the company in 1944 and has been sales manager of Permanente's Pacific Northwest Division since 1946. This position will now be filled by Gordon K. Wilson, formerly assistant Northwest Division sales manager.

ACI Records System Revised

Effective about Oct. 1, 1959, ACI membership records will be converted to the IBM punched card system of record maintenance. This decision by the Executive Committee was prompted to assure speedier and more efficient handling of membership records at less expense.

Under the new system, dues invoices, and JOURNAL and other large mailings will be handled mechanically in a fraction of the time now required.

Should your JOURNAL fail to reach you, or is misaddressed, due to the records changeover, please notify ACI headquarters so prompt correction can be made.

Stress Measurement Symposium, January 25-29

A 5-day symposium devoted to stress measurement is scheduled to be held January 25-29 at Arizona State University, Tempe. Seven leading authorities on stress analysis methods and instrumentation will discuss various methods of stress analysis, selection of methods, possibilities, and limitations. The symposium is being sponsored by *Strain Gage Readings*.

Registration fee for the symposium is $200 per registrant; 25 percent discount for university faculty members. Registration will close Jan. 18, 1960. Brochure and further details available from *Strain Gage Readings*, 5602 East Monte Rosa, Phoenix, Ariz.

OTS begins second year as translation "clearinghouse"

The government's "clearinghouse" for translations of technical and scientific literature recently began its second year of operation.

The Office of Technical Services, U. S. Department of Commerce, reported 4055 complete translations and 35,000 abstracts of foreign technical literature, most of it

Russian, were made available to the public during the program's first year.

OTS was authorized by Congress in July 1958 to collect, catalog, and distribute to science and industry the many thousands of translations done by U. S. government agencies for their own use. Prior to establishment of the OTS clearinghouse, there was no agency through which the material could be released to the public.

Sales of translated documents, which are priced at the cost to OTS of reproduction and handling, exceeded $40,000 during the program's first year, representing the sale of 80,000 documents, according to John C. Green, director of OTS.

In January 1959, OTS, with the cooperation of the Special Libraries Association, began publication of a semi-monthly journal intended to serve as a central source of information in the United States on all new translations available to science and industry. The journal, *Technical Translations*, lists and abstracts not only translations done by the government agencies, but those by private organizations as well. Subscriptions reached 2200 at the end of its sixth month of publication.

Mr. Green also disclosed a new peak in sales volume for a companion, but considerably older, OTS program, the distribution of reports of scientific and technological research done by or for U. S. government agencies, including the Army, Navy, Air Force, and Atomic Energy Commission.

Additional information on the OTS programs may be obtained by writing to OTS, U. S. Department of Commerce, Washington 25, D. C.

Dissolve engineering firm of DeLeuw, Cather and Brill

The engineer-architect firm of DeLeuw, Cather and Brill was dissolved on June 30, 1959, but will continue to function with the present staff to complete all outstanding assignments.

Clinton B. F. Brill and Benjamin Gray have formed the partnership of Brill and Gray to continue the engineering practice of the former partnership in New York City.

DeLeuw, Cather & Co. will maintain an eastern office in Brookline, Mass.

Prestressed Concrete Institute schedules fifth annual meeting

The fifth annual convention and meeting of the Prestressed Concrete Institute will be held November 1-7 at the Deauville Hotel, Miami Beach, Fla.

The technical program will cover all phases of prestressed concrete design, research, production, and application, both in this country and abroad. ACI members Charles C. Zollman of Schupack & Zollman, Newtown Square, Pa. and A. H. Gustaferro, Consumers Co., Chicago, are in charge of the technical program.

Other ACI members participating in the program include: Peter J. Verna, Jr., Concrete Materials, Inc., Charlotte, N. C., president of PCI; Douglas P. Cone, Florida Prestressed Concrete, Inc., Tampa; George W. Ford, R. H. Wright & Son, Ft. Lauderdale; Jacob Frank, Buchart Engineering Corp., York, Pa.; and Roy L. Peck, *Modern Concrete*, Chicago.

Highlight of the event will be a 3-day, post-convention field trip to Havana, Cuba, to view outstanding examples of prestressed concrete.

Stone and Webster promote Doherty

F. Lawrence Doherty, assistant personnel manager of Stone and Webster Engineering Corp. since 1951, has been promoted to personnel manager.

Consolidated Cement merges with General Portland Cement

Consolidated Cement Corp., with plants at Cement City, Mich., North Paulding, Ohio, and Fredonia, Kan., was recently merged into General Portland Cement Co.

In the future the plants at Cement City and North Paulding will operate as the Peninsular Portland Cement Division of General Portland Cement Co., and the plant at Fredonia will operate as the Victor Portland Cement Division of General Portland Cement.

There will be no changes in the Trinity, Signal Mountain, and Florida Divisions of General Portland Cement Co.

POZZOLITH . . . makes good concrete better

Prestressed concrete beams for 6 bridges at Air Academy

PRECAST BRIDGE BEAM is inspected after post-tensioning. Construction of all prestressed POZZOLITH concrete beams by A. S. Horner Construction Co., Contractors, Denver. Bridges built under supervision of the Air Force Academy Construction Agency.

Part of the striking man-made landscape at the new U. S. Air Force Academy are six major concrete bridges—4 highway and 2 railway spans—that required over 2½ miles of prestressed concrete beams.

POZZOLITH concrete was specified and used for all 128 beams. The mix was designed to produce 4% entrained air, 2-inch average slump and high early strength. Compressive strength of 4500 psi—required before stress application—was achieved in 3 to 5 days with POZZOLITH. Seven day strength was approximately 6500 psi and 28-day strength was well above 7000 psi.

Over 750,000 cubic yards of POZZOLITH concrete were used throughout the Academy complex—to best meet a wide range of concrete requirements. POZZOLITH best provides control over three important concrete quality factors: water content, entrained air and rate of hardening—the *key factors* in obtaining uniform, superior quality concrete.

On your concrete projects, the local Master Builders field man will welcome discussing your requirements. Call him in. He's at your service—and expertly assisted by the Master Builders research and engineering staff—unexcelled in the field of concrete technology.

The Master Builders Company, Cleveland 3, Ohio
Division of American-Marietta Company
The Master Builders Company, Ltd., Toronto 9, Ontario
International Sales Department, New York 17, N.Y.
Branch Offices in all principal cities.

LARGEST OF 6 BRIDGES includes five 120-foot spans of prestressed POZZOLITH concrete beams. Bridge deck is 71-feet wide. Architects: Skidmore, Owings & Merrill, Chicago. Engineer: L. Boduroff, Denver. Prestressing: Prescon Corp., Corpus Christi, Texas.

MASTER BUILDERS® POZZOLITH*

**POZZOLITH* is a registered trademark of The Master Builders Company for its concrete admixture to reduce water and control entrainment of air and rate of hardening.*

Reinforced Concrete Pipe Chosen For

This 96" diameter reinforced concrete pipe easily met the rigid requirements of ASTM Specification C76-57T. Tests indicated this reinforced concrete pipe was capable of loads exceeding 100,000 pounds, demonstrating the strength and durability.

S. J. Groves and Sons Company, Minneapolis, Minnesota—General Contractor
Abbott Contractors, Chicago, Illinois—Sub-contractors
Waukesha Cement Tile Company, Milwaukee, Wisconsin—Pipe Suppliers
American Vitrified Products Company, Milwaukee, Wisconsin—Pipe Suppliers
Lock-Joint Pipe Company, South Beloit, Illinois—Pipe Suppliers

Richard I. Bong Air Force Base Runway Drainage

More than 48,000 linear feet of concrete pipe was included in the Corps of Engineers specifications for the Richard I. Bong Air Force Base at Kansasville, Wisconsin. Many sizes were required, the largest being 96" inside diameter. All reinforced concrete pipe was manufactured and pre-tested to meet the rigid requirements of ASTM Specification C76-57T. The 96" pipe was required to meet a loading test of 96,000 pounds. It met this requirement easily and could have withstood loads exceeding 100,000 pounds!

Reinforced concrete pipe is designed in conformance with specifications comprehending the laws of mechanics plus the results of thousands of tests; it is manufactured by men experienced with concrete product production; and it is pre-tested to demonstrate its strength. The pre-testing is done by means of the 3-edge bearing test which is the most severe loading to which the pipe will ever be subjected!

Reinforced concrete pipe installations, in service for many years, are testimony to the strength, durability and watertightness of concrete pipe reinforced with Welded Wire Fabric.

Concrete pipe manufacturers insist on top-quality reinforcing, meeting rigid specifications – that's why so many of them use American Welded Wire Fabric. This quality product, with its machine-made accuracy, assures the proper distribution of steel. It is pre-fabricated from cold-drawn, 60,000 psi yield strength wires. All intersections are electrically welded to assure mechanical anchorage in the concrete. Wire diameters as large as 0.505" on 2", 3" or 4" centers can be furnished. Its rigid specifications assure superior reinforcement.

Write American Steel & Wire, Dept. 9253, 614 Superior Avenue, N. W., Cleveland 13, Ohio for complete technical literature.

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American Steel & Wire Division of United States Steel

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Tennessee Coal & Iron Division, Fairfield, Ala., Southern Distributors
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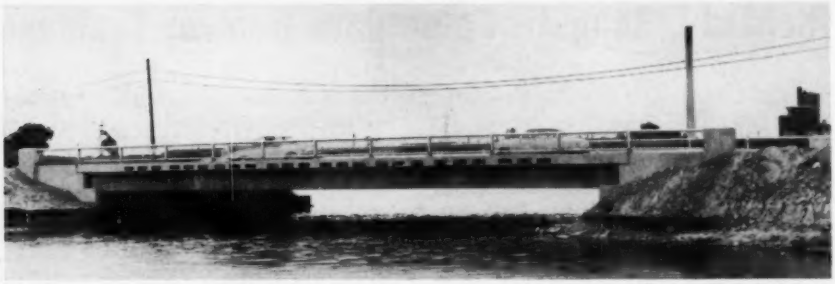
Sections of concrete pipe go in place more quickly and with no job interruptions. Its uniform size insures tight joints and proper alignment.

USS American Welded Wire Fabric, with its small diameter, closely spaced steel members, distributes load stresses evenly throughout the concrete pipe. It is the ideal concrete pipe reinforcement, and can be easily formed to the proper size cage diameters.

American Welded Wire Fabric



Courtesy U. S. Steel Corp.

This four-lane prestressed concrete bridge over the Ottawa River in Toledo, Ohio, was built to replace a two-lane structure on the same site with a minimum of interference with normal traffic. A total of 18 beams were used. Eight formed the outer two lanes which were constructed prior to removing the old bridge. Ten were used to fill the space eft by the old structure.

The beams were pretensioned using 64 strands of seven-wire prestressing steel in the tension zone. Voids in the upper part of the beam were made with fiberboard forms. The upper section was reinforced around the voids. Overnight steam curing followed casting. The beams were precast by Great Lakes Structural Concrete Products, Toledo, and trucked to the site. General contractor was George W. Lathrop and Sons, Toledo.

Ammann and Masters honored by Delaware River Port Authority

ACI members Othman H. Ammann of Ammann and Whitney, New York City, and Frank M. Masters of Masters and Modjeski, Harrisburg, Pa., received stainless steel plaques from the Delaware River Port Authority in recognition of their "imaginative concept and consummate skill in the design and construction of the Walt Whitman Bridge" between South Philadelphia, Pa., and Gloucester City, N. J.

Amrhein appointed PCA structural specialist

The appointment of James E. Amrhein as a structural specialist in the Los Angeles District Office of the Portland Cement Association was recently announced. Mr. Amrhein has been with PCA for the past 5 years, working as a field engineer in the Long Beach and Orange County areas.

Mr. Amrhein holds civil engineering degrees from Manhattan College and Columbia University, both in New York City. He is a member of ACI.

New construction volume to soar past $53 billion

Midyear estimates of new construction volume indicate that in 1959 the value of work put in place will soar past $53 billion according to the July issue of *The Constructor*, official publication of the Associated General Contractors of America. If current expectations are met, the total will be about 10 percent above 1958.

A long-range study made by AGC indicates that the total volume of construction is estimated at above $73 billion for 1959, and is expected to exceed $107 billion in 1970.

Construction equipment distribution curriculum

As a result of the combined effort between education and industry, Clarkson College of Technology, Potsdam, N. Y., will offer a college curriculum leading to a BS degree in construction equipment distribution.

For the past 2 years, the Associated Equipment Distributor's education committee has cooperated with the Clarkson faculty in preparing the curriculum specifically de-

signed to prepare graduates for sales engineering and management positions in the construction industry.

Distributors will provide summer employment for students to afford actual experience to go along with their classroom work.

Further information is available from Prof. Harry J. Waters, Clarkson College of Technology, Potsdam, N. Y.

Blessey completes five-week Latin American lecture tour

Walter E. Blessey, chairman of the department of civil engineering, Tulane University, New Orleans, recently completed a 5-week lecture tour of 12 cities in 11 Latin American nations. The tour was conducted under sponsorship of the Pan American Federation of Engineering Societies.

Professor Blessey spoke to engineering societies and university groups in Cuba, Venezuela, Brazil, Uruguay, Argentina, Chile, Peru, Panama, Costa Rica, El Salvador, and Mexico. He lectured on mass-produced, precast and prestressed concrete structures in the United States, emphasizing all types of construction, including industrial buildings, bridges and off-shore installations.

Professor Blessey is a member of ACI-ASCE Committee 323, Prestressed Reinforced Concrete and ACI-ASCE Committee 326, Shear and Diagonal Tension.

Booker retires from California highway post

B. W. Booker, assistant state highway engineer, who has been responsible for state highway matters in nine San Francisco Bay area counties since 1952 has retired.

Mr. Booker was in charge of the Bay Area Metropolitan Traffic Survey and participated in planning, design, construction, and maintenance of some of the state's most important highways.

J. P. Sinclair, district engineer, District IV, San Francisco, will succeed Mr. Booker.

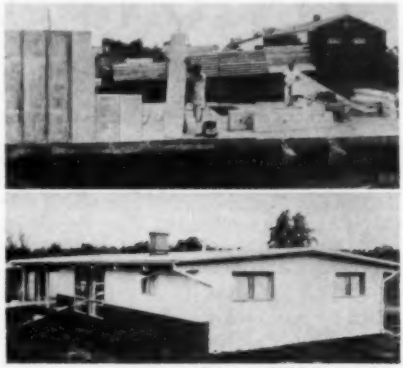
Jellick named consulting engineer for cement agency

J. E. Jellick, manager of the Portland Cement Information Bureau's office, has been appointed a consulting engineer for the bureau. He will continue to serve construction and design people who request information about concrete or inspection of concrete construction jobs.

The bureau is supported by three west coast cement makers: Calaveras Cement Co., Pacific Cement & Aggregates, Inc., both of San Francisco, and Permanente Cement Co., Oakland, Calif.

Swimming pool exposition, December 12-15

The third annual national swimming pool exposition of the National Swimming Pool Institute will be held at the New York Coliseum, December 12-15. The exposition will feature about 200 exhibits on swimming pool design, materials, equipment, and accessories. The third annual NSPI convention, Dec. 11-15, will be held at the Statler Hilton Hotel.



Courtesy CMC Homes, Inc.

CELLULAR CONCRETE HOMES—Precast lightweight concrete panels are being used in Missouri for low-cost housing. Panels 6 ft 8 in. long by 20 in. wide and 3½ in. thick can be handled by two men, who should be able to erect the shell of a six-room house in 70 hr. Ordinary carpenter tools can be used to work the material. Interiors can be painted, plastered, or paneled. The panels, which are patented, have been previously used for industrial applications and are manufactured by Calsi-Crete, Inc., Pacific, Mo.

CLINTON Welded Wire Fabric

A steel backbone for concrete structures

The giant steelman symbolizes the years of experience, top quality materials and rigid quality controls that enable CF&I to produce dependable steel products for all industries. Contractors rely on one of these —CF&I-Clinton Welded Wire Fabric—to add the tensile strength of steel to the durability of concrete.

CF&I-Clinton Welded Wire Fabric is ideal for reinforcing *all* types of concrete—both light and heavy construction. The wire mesh adds years of trouble-free life to the concrete because it helps cushion the shock of heavy loads, distributing the stress over a wide area. The steel reinforcing also minimizes cracking during the setting period because it provides anchorage for the concrete. And the steel fabric holds the concrete tightly together to prevent heaving due to extreme temperature changes.

CF&I-Clinton Welded Wire Fabric is easy to use. The fabric unrolls smoothly and lays flat . . . it can be shaped and cut quickly. It's available in a wide range of gages and meshes for every reinforcing requirement. CF&I-Clinton Welded Wire Fabric is also furnished in mat form to meet individual specifications.

For full details—and prompt delivery—call the nearby CF&I sales office.

FREE! Send for new 32-page catalog, "CF&I Steel Products for the Construction Industry".

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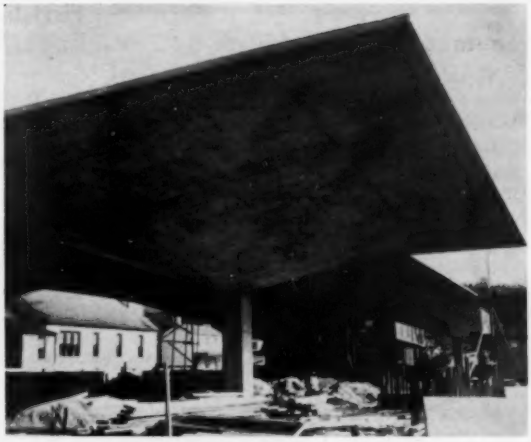

Lightweight concrete umbrellas for both roof and foundations made construction history at All Saints School, Etna, Pa., last year. The hyperbolic paraboloids were an answer to site problems discovered when test borings revealed a 20-ft slag and cinder fill above a water table . . . conditions which limited the building load to 400 psf.

To meet this condition, Pittsburgh architect J. Kenneth Myers, and Triggs and Mellet, design engineers also of Pittsburgh, introduced the umbrella idea. The foundation for 162 x 82-ft building consists of five reinforced concrete hyperbolic paraboloids in conventional position; on these, five columns were cast, four 2 x 2 x 10 ft, and one 2 x 3 x 8 ft. The columns each support an inverted umbrella, the center unit measuring 63 x 57 ft, and four others 58 x 45 ft. The foundation umbrellas are slightly smaller, 34 x 30 ft, and 30 x 25 ft.

Built into each roof section are electric heating coils which will serve in winter to prevent snow loading and accumulation of ice around the mouth of drains. Concrete surface of the roof units was covered with a built-up roofing material, lapped to prevent leaks. The roof drains through each of the supporting columns and through the five foundation piers to underground carry-off lines.

Specifications called for expanded slag aggregate concrete, maximum weight 100 lb per cu ft, and minimum strength of 3000 psi. Some 2200 cu yd of this lightweight material were supplied by the Keystone Division of Dravo Corp., Pittsburgh.

Lapsins joins Judge Engineering

Valdis Lapsins, formerly city plan director of Kettering, Ohio, is now associated with the John W. Judge Engineering Co., also of Kettering.

Penn-Dixie names Petersen industrial relations manager

A. W. Petersen has been appointed manager of industrial relations for Penn-Dixie Cement Corp. according to a recent announcement by M. L. Silcox, vice-president-operations. He succeeds A. H. Bostwick, who is retiring.

Mr. Petersen, who will continue to operate from Penn-Dixie's Nazareth, Pa., offices, has served as assistant manager since joining the company in 1958.

1959-60 Building Research Advisory Board announced

William Gillett, vice-president of Fenestra, Inc., has been reappointed to a second term as chairman of the Building Research Advisory Board.

Three ACI members have been reappointed to the 30-man board: E. K. Abberley, vice-president, Turner Construction Co., New York; A. Allan Bates, vice-president, Research and Development, Portland Cement Association, Skokie, Ill.; and Douglas Parsons, chief, Building Technology Division, National Bureau of Standards, Washington, D. C.

Membership on the board is drawn from the leading scientists, engineers and technologists in government, academic and research institutions, and industry.

Thomas Clark Shedd

Thomas Clark Shedd, professor of structural engineering, emeritus, University of Illinois, Urbana, died July 11, 1959.

Graduating from Brown University, Providence, in 1913, Professor Shedd began his lifetime in education as an assistant in mechanical engineering at Brown. In 1917 he became an instructor in civil engineering at Lehigh University, Bethlehem, Pa. Professor Shedd joined the faculty at the University of Illinois in 1922 as an associate in structural engineering, advancing to the rank of professor in 1934 and serving in that capacity through 1958.

Except for brief intervals in private practice, Professor Shedd spent his life in the field of education. He authored two books: *Theory of Simple Structures* (with Jamison Vawter) published in 1930 and 1941; and *Structural Design in Steel* published in 1934.

A member of ACI and a number of other professional societies, Professor Shedd was past president of the Central Illinois Section of ASCE and the Illinois Engineering Council.

Instrument Society of America plans 14th annual conference

The Instrument Society of America will sponsor its 14th annual instrument-automation conference and exhibit at the Palmer House, Chicago, September 21-25.

Reports on measurement systems being used in research sponsored by the American Association of State Highway Officials at Ottawa, Ill., will be presented at this meeting.

Errata

The following correction should be made in "Commentary on Concrete" by Robert F. Legget, which was published in the March, 1959, JOURNAL.

p. 928—reference is made to the Dome of the Public Library in Melbourne and attributes its design to John Monash. Our attention has been called to the fact that while a design was made by Sir Monash, it was not used. The structural design should be attributed to Trussed Concrete Steel Co., London.

First Industrial Building Exposition, December 1960

New York Coliseum will be the site of the first Industrial Building Exposition and Congress, December 12-15, 1960. The board of sponsors is comprised of 28 of the nation's foremost architects and industrial executives versed in the fields of management, construction, and plant-operation.

The congress will consider such subjects as planning new construction, financing, modernization of older buildings, new types of construction, layout of equipment, construction materials, and the like. Exhibits will be directed to similar themes.

Sauer affiliates with architectural firm

David A. Sauer, formerly assistant professor of architecture, University of Illinois, Urbana, is now principal in the firm of Sauer, Matson and Sanner, architects-engineers, Fort Wayne, Ind.

Ford, Bacon and Davis open new Pacific Coast offices

Charles C. Whittelsey, president of Ford, Bacon and Davis, Inc., engineers and business consultants, New York, recently announced the appointment of John G. Lewis as manager of the firm's new Pacific Coast offices in San Francisco. The company also maintains a Midwest office in Chicago, with construction headquarters at Monroe, La.

Dundee Cement adds two sales representatives

Homer C. Bohn and Miles Merwin have been appointed sales representatives for the Dundee Cement Co. in the Detroit area.

Mr. Bohn served as a civil engineer with the Michigan State Highway Department following his graduation from the University of Detroit. He was later sales engineer with American Prestressed Concrete, Inc., Centerline, Mich. Mr. Bohn is an ACI member.

Mr. Merwin, Ohio State University graduate, was formerly associated with Detroit Fuel and Supply, and Merwin Building Materials Co.

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Durability
Factor: 100

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1 : 2 : 4 Mix

Durability
Factor: 234

WITH PLACEWEL
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Durability
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The above photos show the results of 21 freeze-thaw cycles in a 5% calcium chloride solution. Under standard A.S.T.M. procedures, Placewel concrete had 3½ times greater resistance than plain concrete and was 1½ times better than straight air-entrained concrete.

Liquid Placewel contains air-entraining and dispersing-water reducing agents, *plus* a catalyst to step up hydrolysis and hydration of cement. By breaking-up cement flocs, it releases water for lubrication which would not otherwise be available . . . requires less water in a mix without relying on air-entrainment alone.

You get all the advantages of entrained air without its adverse effects on strength.

Less than 3 oz. Placewel/sack portland cement will—

- increase strength 30% or more (after 28 days' curing)
- improve quality at no extra material cost
- increase workability and placeability
- reduce bleeding and segregation
- give controlled air entrainment
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JOHNS-MANVILLE PRODUCTS

JOHNS-MANVILLE

McCall joins Lock Joint Pipe research staff

John T. McCall has been appointed assistant director of research and development of Lock Joint Pipe Co., East Orange, N. J.

Graduated from Michigan College of Mining and Technology in 1947 with a BS in civil engineering, Dr. McCall received his MS from Michigan State University in 1951, and his PhD from Purdue University in 1956. Active in the fields of teaching, research, and industrial consulting, he has recently been on the faculty of Michigan State University as associate professor in the Department of Applied Mechanics.

Dr. McCall is the author of papers on fatigue in plain concrete, the effects of calcium chloride in concrete, and lightweight aggregates. He is a member of ACI and at the present time a member of ACI Committee 215, Fatigue of Concrete.

Ehrenburg moves to United Concrete Pipe

Otto Ehrenburg, formerly concrete technologist with Pacific Gas and Electric Co., is now research laboratory engineer with United Concrete Pipe Corp., Baldwin Park, Calif. Prior to PG&E, Mr. Ehrenburg was an engineer with the U. S. Bureau of Reclamation for many years.

Cutts appointed to NSF advisory panel

The National Science Foundation, Washington, D. C., has recently appointed Charels E. Cutts, head of the department of civil engineering, Michigan State University, East Lansing, to serve as a member of its advisory panel for engineering sciences.

Dr. Cutts is currently serving on two ACI committees: 115, Research, and 215, Fatigue of Concrete.

LOOKING AHEAD

Oct. 11-16, 1959—3rd Pacific Area National Meeting, American Society for Testing Materials, Sheraton-Palace, San Francisco, Calif.

Oct. 19-23, 1959—47th Annual National Safety Congress and Exposition, Conrad Hilton, Congress, Sheraton, Morrison, and LaSalle hotels, Chicago, Ill.

Oct. 29-30, 1959—42nd Annual Meeting, National Slag Association, Boca Raton Hotel and Club, Boca Raton, Fla.

Nov. 1-7, 1959—5th Annual Convention, Prestressed Concrete Institute, Deauville Hotel, Miami Beach, Fla.

Nov. 2-5, 1959—Regional Meeting, American Concrete Institute, Hotel Continental Hilton, Mexico City, Mexico

Nov. 17-19, 1959—Fall Conference, Building Research Institute, Shoreham Hotel, Washington, D. C.

Jan. 25-29, 1960—Stress Measurement Symposium, Arizona State University, Tempe, Ariz.

Feb. 1-5, 1960—Committee Week, American Society for Testing Materials, Hotel Sherman, Chicago, Ill.

Feb. 15-19, 1960—Annual Convention and Exposition, National Sand and Gravel Association, Conrad Hilton Hotel and Coliseum, Chicago, Ill.

Feb. 22-24, 1960—Annual Convention and Exposition, National Crushed Stone Association, Conrad Hilton Hotel, Chicago, Ill.

Mar. 14-17, 1960—56th Annual Meeting, American Concrete Institute, Commodore Hotel, New York, N. Y.

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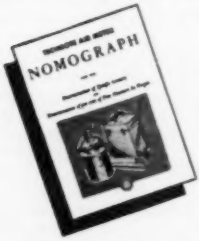
FIRST CHOICE AROUND THE WORLD

The Techkote Air Meter now provides a multi-range testing instrument that vastly increases the usefulness of the air meter. It is now possible to measure accurately—within a fraction of a degree—entrained air with one instrument on numerous materials such as lightweight concrete, mortar, plaster and soil, in addition to regular concrete.



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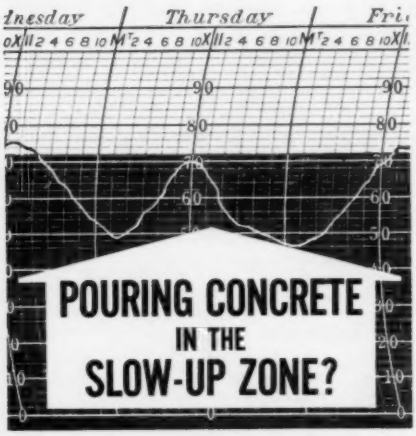

Have your ready-mix supplier add SOLVAY CALCIUM CHLORIDE to avoid costly delays!

When temperatures vary from 70° down to 50°—it spells "SLOW-UP" which means costly delays in your concreting operations. This drop below 70° sharply decreases strength development and lengthens the waiting period before finishing.

A drop from 70° to 50°, for example, cuts 3-day strength up to 40%. To prevent this, add a low-cost 2% of SOLVAY Calcium Chloride to your concrete.

With this acceleration, concrete poured at 50° has a 3-day strength up to 40% *greater* than ordinary concrete cured at the ideal temperature of 70°!

When you use SOLVAY Calcium Chloride, you do away with overtime finishing, delays in form removal, delays between operations. You save up to 50% on protection time. And you get *better* concrete—increased in *both* early and ultimate strength—with lower water-cement ratio for more moisture-and-wear-resistant concrete.

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Honor Roll

January 1—August 31, 1959

Our Honor Roll this month lists 233 member names representing 2.3 percent of ACI's total membership (9965) who have been "member conscious" this year. Add your name to the "Honor Roll" by securing new members. The membership committee recently inaugurated the following plan for crediting individuals for obtaining new members: 1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

Samuel Hobbs	44
Robert P. Witt	40½
George B. Southworth	23
Alfonso Marin E.	15
Louis A. Gottheil	13
David A. Sauer	13
Faraj Tajirian	13
Robert F. Adams	12
James Chinn	12
Charles C. Luther	10
Newlin D. Morgan, Jr.	10
H. C. Pfannkuche	10
Chas. W. Cole, Jr.	9
Phil M. Ferguson	9
Kenneth M. Huber	9
A. T. Klassen	9
Francesco Sardella	9
Joseph J. Shideler	8½
Joe W. Kelly	8
Glenn C. Thomas	7
Henry Aaron	6
Jose Maria Bravo	6
John L. Burdick	6
S. D. Burks	6
Ward Dobbin	6
Alberto Dovali Jaime	6
Myron L. Goral	6
Mauro N. Guzman	6
John E. Heer, Jr.	6
Gregorio Hernandez	6
Horace G. Hil, III.	6
Aleck E. Hiscox	6
Truman R. Jones, Jr.	6
Elias Kardaras	6
George J. Kerekes	6
George E. Large	6
Leo M. Legatski	6
George N. Scofield	6
L. J. Sewell	6
Ferruh Taskin	6
John P. Thompson	6
George Batievsky	5
W. S. Cottingham	5
Victor M. Gallo	5
Jack C. McCoe	5
Abel Moreno Przespolewski	5
W. E. Moulton	5
Gene M. Nordby	5
Rafael Enrique Pacheco	5
C. P. Siess	5

Michael Alexander..... 4
W. H. Armstrong..... 4
Ira M. Beattie..... 4
George P. Duecy..... 4
Eberhart Gunther..... 4
Martin J. Gutzwiller..... 4
B. E. Kester..... 4
S. J. Moore..... 4
Wendell H. Nedderman.... 4
Constantin Polidoroff 4
Jerome M. Raphael..... 4
Emilio Rosenblueth..... 4
Felix R. Sarapu..... 4
M. F. A. Siddiqui..... 4
Howard Simpson..... 4
James E. Stanners..... 4
Lewis H. Tuthill..... 4
Victor Achim..... 3
John H. Adams..... 3
Mihran Agbabian..... 3
Richard G. Allen..... 3
Jay B. Ames..... 3
William C. Alsmeyer..... 3
Tamnoon Ansussinha 3
John D. Antrim..... 3
Edward E. Baker..... 3
Luther E. Bell..... 3
O. R. Bell..... 3
Dutton Biggs..... 3
J. G. Bodhe..... 3
D. E. Bowen..... 3
Dan E. Branson..... 3
John E. Breen..... 3
Ralph B. Brenan, Jr..... 3
C. F. Brown, Jr..... 3
Theodor H. Busck..... 3
G. M. Butcher..... 3
Guillermo Castellanos G.... 3
T. J. Cavanagh..... 3
Clinton H. Chalmers..... 3
Solomon Chornik S..... 3
Marvin B. Cohen..... 3
Ralph G. Crimms, Jr..... 3
M. H. Cutler..... 3
Raymond E. Davis..... 3
Roger D. de Cossio..... 3
Ray C. Dickerson..... 3
O. A. Dresser..... 3
Frederick W. Drury, Jr..... 3
Thomas A. Duwelius..... 3
Gene E. Ellis..... 3
B. A. Eskenazi..... 3
R. H. Evans..... 3
Russell S. Fling..... 3
James Lee Ford..... 3
Ray J. Foss..... 3
Joseph J. Fox..... 3
Russell H. Fuller..... 3
Junius R. Gardner..... 3
Ben C. Gerwick, Jr..... 3
E. J. Gianotti..... 3
Raleigh DeVisme Gipps.... 3
Elliot A. Haller 3
George F. Hammersmith... 3
Robert B. Harris..... 3
Eric J. Hayford..... 3
Howard H. Hays..... 3
F. J. Hosgood..... 3

Samuel I. Iwata..... 3
J. E. Jellick..... 3
J. E. Jones..... 3
William W. Karl..... 3
Joseph Karni..... 3
Allan A. Kay..... 3
R. Evan Kennedy..... 3
Frank Kerekes..... 3
Narbey Khachaturian..... 3
Frank R. Killinger..... 3
H. I. King..... 3
Solomon Kirschen..... 3
Erik Kolle..... 3
William C. Krell..... 3
Felix Kulka..... 3
Robert R. Kuske..... 3
Edward Laing..... 3
Ronald Lazar..... 3
Elmer C. Lee..... 3
Janis Liepins..... 3
Henry Lipkind..... 3
Russ A. Loveland..... 3
Pedro Lainez Lozada C.... 3
George R. McCaulley..... 3
Douglas McHenry..... 3
Keith E. McKee..... 3
Ernst Maag..... 3
M. F. Macnaughton..... 3
Frank B. May..... 3
Amos R. Mead..... 3
Eduardo Mercado Flores... 3
L. Boyd Mercer..... 3
Gustaf Mickos..... 3
Otto H. Monch S..... 3
Almer F. Moore..... 3
Joseph H. Moore..... 3
William J. Moore..... 3
Gene R. Morris 3
Leslie E. Moss..... 3
Donald H. Olson..... 3
Cenap Oran..... 3
Diego Parra Pardi..... 3
Orris O. Pfutzenreuter..... 3
R. C. Postlethwaite..... 3
Yohanan Priell..... 3
Jack L. Randall..... 3
E. B. Rayburn, Jr..... 3
Thomas J. Reading..... 3
Raymond C. Reese..... 3
Ricardo Rivas-Roman..... 3
Sigmund Roos..... 3
Antonio I. Rosquete..... 3
Donald A. Sawyer..... 3
Herbert M. Schwartz..... 3
Burhl M. Scruggs..... 3
Paul Serfass..... 3
Leo G. Shea..... 3
L. Shector..... 3
Jehangir C. Shroff..... 3
Floyd O. Slate..... 3
Robert W. Smith..... 3
Stuart H. Snyder..... 3
Ralph W. Spencer..... 3
Leroy A. Staples..... 3
W. N. Steinmann..... 3
Willard Stevenson..... 3
Anthony A. Styner..... 3
T. Tassios..... 3

F. D. Taylor..... 3
Pastor B. Tenchavez..... 3
T. W. Thomas..... 3
J. Antonio Thomen..... 3
Sophus Thompson..... 3
Sylvester J. Turley..... 3
Ellis S. Vieser..... 3
Bernardo Villegas..... 3
L. T. Willoughby..... 3
Vernon S. Winkel..... 3
Roberto Zepeda Aldana... 3
Stanley G. Zynda..... 3
Pedro M. Bassim..... 2
Walter E. Blessey..... 2
Boris Bresler..... 2
Paul E. Chen..... 2
P. Q. Chin..... 2
Miles N. Clair..... 2
George A. Dinsmore..... 2
Haraldo Duarte Villela.... 2
H. H. Edwards..... 2
Ambrosio R. Flores..... 2
D. A. Guntin..... 2
K. Hajnal-Konyi..... 2
John P. Hollings..... 2
Simon Lamar..... 2
Oscar Lattore M..... 2
Joachim F. Leppmann..... 2
Leo Liberthson..... 2
Carl F. Long..... 2
Patrick McNally..... 2
Ernest C. Marmet..... 2
Howard R. May..... 2
Jorge Rodriguez Lebron.... 2
Sabri Sami..... 2
Nicanor B. Santos..... 2
Charles Ream Sargent..... 2
Herbert A. Sawyer, Jr..... 2
Robert F. Schoening..... 2
Javier Santolalia Silva..... 2
K. Sitaraman..... 2
Jeno F. Toppler..... 2
A. M. Tovar..... 2
Jose R. Perez Valentin..... 2
C. K. Wang..... 2
J. H. Weaver..... 2
Richard E. Woodring..... 2

ADD

YOUR NAME

to the

ACI

HONOR ROLL

This is to Certify that

John Smith

is a

Member

of this institute and fully entitled to the privileges granted by its Charter and Bylaws

_____
President

_____
Secretary-Treasurer

Date of membership: July 1, 1952

Above is shown the membership certificate now available to all interested members. (Student members become eligible for certificates when they advance to one of the higher membership grades.) The actual certificate is 10 x 12 in. on parchment paper suitable for framing.

Return this order form together with your remittance—to avoid the added expense of billing—and your certificate will be mailed to you. Please allow 90 days for preparation of the certificate.

Please accept my order for a membership certificate at the price of $3.00.

☐ Check ☐ M.O. Enclosed

My name is to appear exactly as follows:

_____
(Print or type)

Signature_____

New Members

The Board of Direction approved applications in the following categories: 61 Individual, 2 Corporation, 10 Junior, and 8 Student, making a total of 81 new members. Considering losses due to deaths, resignations and nonpayment of dues, the total membership on July 1, 1959 was 9965.

Individual

ALDRED, PETER NORMAN, Perth, West Australia (Governing Dir., Cement Aids)
AYRES, RAYMOND A., Fanwood, N. J. (Vice-Pres. & Mgr., Plainfield Lab., The Haller Testing Labs.)
BAILEY, RICHARD B., Camas, Wash. (Mgr., Chem. Sales, Chemical Prods. Div., Crown Zellarbach Corp.)
BAILEY, SHELLY N., Sacramento, Calif. (Techn., Calif. Dept. of Water Resources Concrete Lab.)
BAKER, JULIUS O., La Mirada, Calif. (Sales Engr., L. M. Scofield Co.)
BARNARD, PHILIP D., Houston, Tex. (Pres. & Gen. Mgr., Southern Inspection Service, Inc.)
BARNEA, IDA, Holon, Israel (Deputy Chf. Struct. Engr., P. W. D., Headquarters)
BARNOFF, ROBERT M., University Park, Pa. (Asst. Prof. of Civil Engrg., The Pennsylvania State Univ.)
BAUER, FRED J., Trident, Mont. (Chf. Chem., Ideal Cement Co.)
BELCHER, LLOYD R., Birmingham, Ala. (Chf. Struct. Engr., Rust Engrg. Co.)
BORN, HARRY, Superior, Neb. (Chf. Chem., Ideal Cement Co.)
BRASWELL, ALBERT H., JR., Charlotte, N. C. (Field Engr., Penn-Dixie Cement Co.)
BRYANT, FRANK G., Austin, Tex. (Cons. Engr.)
BURKE, EDWARD, Greenhithe, Kent, England (Dir. of Research, Associated Portland Cement Mfrs., Ltd.)
CLARK, LEWELL H., Jackson, Wyo. (Jr. Engr., Harold S. Clark)
CLARKE, JOSEPH R., Montreal, Canada (Constr. Engr., Montreal Engrg. Co. Ltd.)
COHAN, H. R., Dunedin, Fla. (Constr. Insp., Security Federal Savings & Loan Assn.)
CONNER, HAROLD W., Chicago, Ill. (Struct. Engr., PCA)
COOK, HERMON C., JR., Memphis, Tenn. (Mgr., Southern Lift Slab, Inc.)
CRANE, GEORGE B., Sacramento, Calif. (Asst. Physical Testing Engr., State of Calif., Dept. of Water Resources)
CURTIN, EDWARD J., Arlington, Calif. (Sales Repres., Owl-Service Rock Co.)
CYR, GUY, Montreal, Canada
DAS, BHUBAN MOHAN, Balasora, Orissa, India (Asst. Engr., Gov't of Orissa)
ESCHENBRENNER, H., Columbus, Ohio (Vice-Pres., Arrow Universal Concrete Div., Marble Cliff Quarries Co.)
ESKESTRAND, DEL, Havre, Mont. (Partner in Constr. Firm)
FIENE, CHARLES R., Elmhurst, Ill. (Engr., Elmhurst Chicago Stone Co.)
FREEMAN, HAROLD C., Whittier, Calif. (Sales Mgr., L. M. Scofield Co.)
GEIGER, JACK SYLVESTER, Pomona, Calif. (Constr. Insp., Pasadena City School District)
GRAATS, OSWALD, Johannesburg, South Africa (Cons. Civil & Struct. Engr.)
HALL, FRANCIS E., Greenville, Miss. (Sr. Partner, Hall & Norwood, Archs. & Engrs.)
HANFT, JOHN D., Philadelphia, Pa. (Sr. Engr., Head of Engrg. Dept., Turner Constr. Co.)
HANSEN, RONALD P., Perth, West Australia (Dir., Cement Aids & Mgr., Western Australian Br.)
HARDY, C. M., Odessa, Tex. (Struct. Cons. & Engr.)
HONEY, FRANCIS E., Portland, Ore. (Struct. Engr.)
JANSSONIUS, G. F., Amsterdam-Z Holland (Chf. Engr., Bridge Dept., Pub. Wks., Amsterdam)
LAYER, ROBERT W., JR., Elgin, Ill. (Partner, Struct., Engr. & Arch., Schmidtke & Layer)
LIVINGSTON, JOHN STANLEY, Montreal, Canada (Sales Repres., Dewey & Almy Constr. Chem. Div., W. R. Grace & Co. of Canada, Ltd.)
MEDIAVILLO, FELIXBERTO, Manila, Philippines (National Planning Comm.)
MEIKEL, E. J., JR., Ft. Collins, Colo. (Chf. Chem., Ideal Cement Co.)
MICHALEK, KURT J., Don Mills, Ont., Canada (Designer, T. Eaton Co., Ltd.)
MOORES, WARREN L., Nevada City, Calif. (Proj. Mgr., Stolte, Inc.)
NIETO-CASAS, LEOPOLDO, Mexico D. F., Mexico (Struct. Design)
ORTIZ, FRANCISCO R., Mexico D. F., Mexico (C. E.)
OTIS, DONALD W., Hutchinson, Kan. (Engr., Chalmer & Borton)
PETERS, HUGH F., Phoenix, Ariz.
PHIN, JAMES P., Hamilton, Canada (Vice-Pres. & Gen. Mgr., Mixed Concrete Supply Ltd.)
PUTNAM, CLAY A., Lubbock, Tex. (Contractor)
RAHMAN, REZAUR, Azimpura, Dacca, East Pakistan (Research Asst., Bob M. Galloway, C. E. Dept., A & M College)
REYNA, CARLOS, Caracas, Venezuela (Rector, Universidad Catolica Andres Bello)
ROWLETT, LLOYD E., Florence, Colo. (Chf. Chem., Ideal Cement Co.)
RUSSELL, JOHN E., Agana, Guam (Gen. Mgr., Hawaiian Rock Products, Inc.)
SEELEY, GEORGE F., Fullerton, Calif. (Engrg. Physics Instructor, Cerritos College)
STEWART, JACK F., Tulsa, Okla. (Asst. Mgr., Tulsa Br., United States Testing Co.)
SYLVANUS, ALFRED, Los Angeles, Calif. (Cons. C. E.)
STRINGHAM, EDWARD B., Albany, N. Y. (Engr., Penetryn System, Inc.)
THOMSON, MISS R. J., Syracuse, N. Y. (Pres. & Gen. Mgr., Thomson Form Hardware Co.)
VARMAZIS, NIKOS, Toronto, Canada (C. E.)
WAYMIRE, R. CHARLES, Omaha, Neb. (Struct. Engr., Leo A. Daly Co.)
WILSON, JAMES DAVID, Wilmington, Del. (Princ. Arch. & C. E., E. I. du Pont deNemours & Co.)
WILSON, THOMAS GERALD, Kingston, Canada (Concrete Insp., Dept. of Hwys., Ont.)
WIREUS, EUGENE ANTHONY, Tujunga, Calif. (C. E., PCA)

Corporation

HUNT PROCESS CO., INC., Los Angeles, Calif. (John G. Kasnetsis, Sales Engr.)
TERMINAL CONSTR. CORP., Wood-Ridge, N. J. (Norman Weiss, Chf. Engr.)

Junior

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STILLWELL, JAMES D., Champaign, Ill. (Univ. of Ill.)
ZAGUSTIN, ELENA Caracas, Venezuela (Univ. Central de Venezuela)

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Junior beams used as construction joints

A major problem for warehouse operators is the deterioration of concrete flooring at the construction joints.

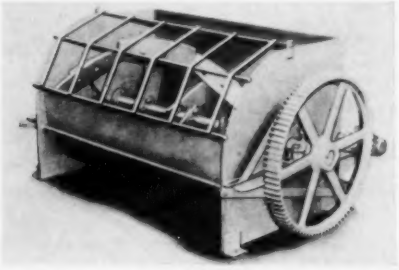
D. D. Davis Construction Co., Youngstown, Ohio, believes it has solved the problem by using Junior Beams, lightweight steel structurals made by Jones & Laughlin Steel Corp., as combination construction-contraction joints. The top flange of the beam, by permitting almost an inch of contraction before the concrete edges are exposed, will substantially reduce crumbling and splitting caused by loads from wheels of materials handling equipment. According the company this method also has construction advantages: serves as built-in screeds and bulkheads; eliminates the temporary wood set-ups required for each bay.—Jones & Laughlin Steel Corp., 3 Gateway Center, Pittsburgh, Pa.

Heavy duty masonry saw

Ty-Sa-Man Machine Co. announces the development of a heavy duty masonry cut-off saw designed and built to carry 5 hp motor and 22 in. diamond blades. The model is also available with 3 hp motor and 16 in. blade guard for marble. Either model is available for mounting over conveyor or table, or as complete portable package. Literature and price list available. —Ty-Sa-Man Machine Co., 1043 White Ave., Knoxville, Tenn.

Concrete mixers

A group of new, standard cement mixers featuring mixing bowls with heavy liners, which are quickly renewable when worn, is in production at General



Engines Co. For durability, the paddles are easily adjustable to give the best and fastest mixing with minimum power according to company engineers.

Dry batch construction mixers are available in sizes from 15 to 60 cu ft capacity. Other sizes can be built to meet specifications. Literature available. —General Engines Co., Route 130, Thorofare, N. J.

Modified finisher

Adapted for off-the-job travel, the finishing machine pictured below is doing double duty, spreading and

finishing, during concreting of aircraft taxi and parking areas at Malton Airport, near Toronto, Ont., Canada.

Huron Construction Co., Ltd., Chatham, Ont., modified the Blaw-Knox finisher to include two 45-in. diameter rubber-tired wheels, which lower for tow-type travel between paving strips, eliminating trailer loading and unloading of the unit for job-to-job hauling.—Blaw-Knox Co., 300 Sixth Ave., Pittsburgh, Pa.

Plastic paper for concrete curing

White plastic-coated reinforced paper has been developed by American Sisalkraft Corp. for curing concrete highways and airports. Stronger curing blankets and easier handling are features claimed by the manufacturer. Available in standard widths from 9½ in. to 25 ft.—American Sisalkraft Corp., Attleboro, Mass.

Rubber gasket for recessed pipe

C-R Tylox gaskets for recessed sewer pipe are the latest addition to the Hamilton Kent line of flexible gaskets. They are of "snap-on" type, designed for use with recessed tongue-and-groove or bell-and-spigot pipe of all sizes, and may be made of either rubber or neoprene. Engineering data, diagrams, and installation information are available.—Hamilton Kent Manufacturing Co., 427 West Grant St., Kent, Ohio

Pneumatic demolition tool

Engineered Equipment, Inc., has introduced a pneumatic tool engineered for heavy demolition of masonry, concrete, stone, and asphalt surfaces.

Little recoil and low air consumption are features of the Model CL 34 cited by the manufacturer. Simple lever mechanism enables the operator to change chisels and spades. All shaped parts are forged for extra strength and long life.—Engineered Equipment, Inc. Waterloo, Iowa.

Tandem-axle trucks

A complete new line of White tandem-axle construction trucks for use with concrete mixers, dump bodies, dump trailers, flatbed bodies, and other specialized operations are offered in four model designations, both gas and diesel, from 35,000 to 75,000 lb gvw. A wide range of component options permits tailoring the chassis, engine, transmission, and axle combinations of each model to exact requirements of the operator according to the manufacturer.

For power take-off applications, all models have adequate capacity for front engine drive, SAE side mount, extended counter loft, or midship auxiliary and power tower installation.—The White Motor Co., 842 E. 79th St., Cleveland, Ohio

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

GLEN CANYON DAM FILM—"Canyon Conquest," 16-mm sound and color film from the Bureau of Reclamation, tells the story of the first 2 years of construction at the Glen Canyon Dam site. The 20-min film is available for showing to any group. There is no service charge, but the user must pay the return postage.—Office of the Commissioner, Bureau of Reclamation, Washington, D. C., or Regional Director, Bureau of Reclamation, Salt Lake City, Utah

BE READY FOR READY-MIX CONCRETE—Sets forth the "do's and don't's" about working with ready-mixed concrete—particularly for small concrete projects such as walls, walks, driveways, and patios. Booklet explains and illustrates layout, forming, estimating quantities, placing, screeding, floating, surface finishing, and curing procedures.—Permanente Cement Co., Kaiser Building, 1924 Broadway, Oakland 12, Calif.

REDISPERSIBLE POLYVINYL ACETATE POWDER—5-page technical bulletin on Vinac RP-250 redispersible polyvinyl acetate powder. Concrete block fillers, joint cements for dry wall construction, spackling compounds, and powder paints are types of dry mix products now being formulated with Vinac RP-250 powder.—Colton Chemical Co., Division of Air Reduction Co., Inc., 1747 Chester Ave., Cleveland 14, Ohio

TRUCO PRODUCT INFORMATION SHEET—Describes new developments in Truco portable and mobile diamond drilling equipment. Describes the Truco Mighty Model G, a heavy-duty portable diamond drilling machine for drilling holes up to 20-in. OD through reinforced concrete, terrazzo, tile, marble, brick, asphalt, etc.—Truco Masonry Drilling Division, Wheel Trueing Tool Co., 3400-342 West Davison, Detroit 38, Mich.

KURE-N-SEAL (BP 1071)—Presents application and test data on Kure-N-Seal for curing, sealing, and dustproofing newly-placed concrete surfaces in one operation. Product is a blend of synthetic rubber resins in fast evaporating solvents, which produces a transparent, hard glossy film to protect the floor from traffic abrasion, water spillage, mild acids, and alkalies.—L. Sonneborn Sons, Inc., Building Products Division, Dept. K, 404 Fourth Ave., New York 16, N. Y.

ATLAS BULLETIN NO. 5-2—Revised bulletin describes Atlas acid-proof cements and their various recommended applications. Also contains a resistance characteristics chart.—Atlas Mineral Products Co., Mertztown, Pa.

1959 PRICE LIST AND INDEX OF AMERICAN STANDARDS—Lists the more than 1800 American Standards approved to date by the American Standards Association. A few of the many fields covered include: automotive, chemical, civil engineering and construction, ferrous materials and metallurgy, materials handling, highway safety, and consumer goods. Also included in the publication are the international standards recommendations published by the International Organization for Standardization and the International Electrotechnical Commission.—American Standards Association, 70 East 45th St., New York 17, N. Y.

SPANBLOK ROOF AND FLOOR SYSTEM—Catalog describes Spanblok prefabricated prestressed concrete plank for use in roofs and floors. Units are fabricated in depths of 4, 6, and 8 in. with a modular width of 16 in. and spans up to 32 ft.—Spanblok Division, Prestressing Inc., P. O. Box 10368, San Antonio 21, Tex.

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ARMY RESEARCH SUMMARY—Army research and development projects in the major fields of science and technology under way as of September, 1958, are reviewed in a six volume report, *Annual Research Task Summary—1959 Edition.*

A brief description—purpose, scope, approach, and status—is given for each task. Agencies or contractors are identified along with principal investigators involved in each project.

Of particular interest are the sections on civil engineering in V. 5, "Geophysical Sciences and Engineering" (483 pp., Order No. PB 151498, $4.75) and the section on nonmetallic materials technology in V. 6, "Materials Technologies" (271 pp., Order No. PB 151499, $2.75).—Office of Technical Services, U. S. Department of Commerce, Washington 25, D. C.

SUPREME STRAND CHUCKS WALL CHART—19 x 25-in. chart gives service information on Supreme strand chucks used to end-anchor prestressing wire strand. Includes simple sketches of all phases of strand chuck use and maintenance.—Supreme Products Corp., 2222 S. Calumet Ave., Chicago 16, Ill.

CONCRETE FACTS (P-36B)—Six-page bulletin contains a concise analysis of the nine vital factors governing the production of high quality concrete. Data are given on effect of Pozzolith on concrete flexural, bond, and compressive strength; volume change; durability; retardation; permeability; workability; and scale resistance.—The Master Builders Co., Cleveland 3, Ohio

RUBBER PADS FOR BRIDGE BEARINGS—Illustrated folder, compiled to furnish latest, detailed, useful data on Herco neoprene absorbent bridge and industrial bearing pads.—Herco Rubber Crafters, Division of Hecht Rubber Co., Inc., 482-484 Riverside Ave., Jacksonville 2, Fla.

MINIMUM STANDARDS FOR RESIDENTIAL POOLS—Revised recommended standards recently approved by the National Swimming Pool Institute presents standards of construction and design presently considered minimum requirements consistent with health, safety, and public welfare.—Robert S. Greene, National Swimming Pool Institute, Harvard, Ill.

SF INTERLOCK PAVING STONES—Folder illustrates and describes S-shaped interlocking paving stones which have been in production for the past 2 years in a concrete works in Germany. Manufacturer states that SF-Bound Paving Stone has found considerable interest with local public works administrations. Manufacturer would like to contact concrete firms in the United States interested in a franchise agreement.—Allgemeine StraBenbaustoff-Gesellschaft m. b. H, Unter den Linden 33, Bremen-St. Magnus, Germany

MBSA REPORTS—A new publication by the newly formed Modular Building Standards Association, Washington, D. C. The publication will be distributed to all MBSA members, and is a compilation of current technical articles on modular progress; news and status of ASA's recently reconstituted Committee A62 on dimensional standards for building products; and 2- and 4-page technical brochures on new modular products being introduced by MBSA's manufacturer-members.—Modular Building Standards Association, 2029 K St., N. W., Washington, D. C.

CASE STUDIES ON DAMS (Bulletin MBR-P3)—48-page report features the role played by Pozzolith as a concrete improving admixture in the construction of 25 major dams built in the United States, Japan, Canada, and South America.—The Master Builders Co., Cleveland 3, Ohio

ELLIPTICAL CONCRETE PIPE FOR SEWERS AND CULVERTS—Fully illustrated, 4-page brochure explaining the use of elliptical pipe to obtain round pipe flow equivalents. Diagrams, data charts, and tables fully describe elliptical pipe sizes, and compute discharge flow rates for the full range of pipe sizes.—United States Concrete Pipe Co., 1500 Union Commerce Bldg., Cleveland 14, Ohio

ECONOMY THROUGH COATED, NONBONDING TENDON—Describes the features of coated nonbonding tendons for post-tensioned concrete. Shows pictures and diagrams of tendon placement in forms and how Prescon tendons are coated and wrapped to protect more effectively against corrosion and reduce friction in tensioning.—The Prescon Corp., P. O. Box 4186, Corpus Christi, Tex.

REUSABLE INFLATABLE VOID FORMS (Catalog No. 500)—Describes and illustrates the process of forming a void in concrete with reusable rubber forms. Used for monolithic sewers, monolithic pressure pipe, drains, cast-in-place cored roof and floor slabs; bridge decks, etc., as well as precast products such as hollow piles, cored columns, and piers.—Elgood Concrete Forms Corp., 378 Ten Eyck St., Brooklyn 6, N. Y.

THREE BASIC TESTS FOR CONCRETE QUALITY CONTROL—Folder shows how the slump test, air entrainment determination, and testing of concrete cylinders is performed. Photographs illustrate the major steps in each testing operation.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

TRETOL MANUAL—Detailed information on a line of more than 33 admixtures and surface treating products for concrete and masonry construction and building maintenance. Manual includes data on formula of each product, where it may be used, its advantages, how it works, what it does, and methods of application.—Tretol, Inc., 6531 West 63rd St., Chicago 38, Ill.

VIBRATORY FEEDERS—32-page catalog contains complete descriptions, data, and specifications for Syntron's complete line of six small-capacity, four heavy-duty, and three extra-heavy-duty standard electro-magnetically vibrated feeders. Illustrates many typical installations and gives a variety of application diagrams.—Syntron Co., 1471 Lexington Ave., Home City, Pa.

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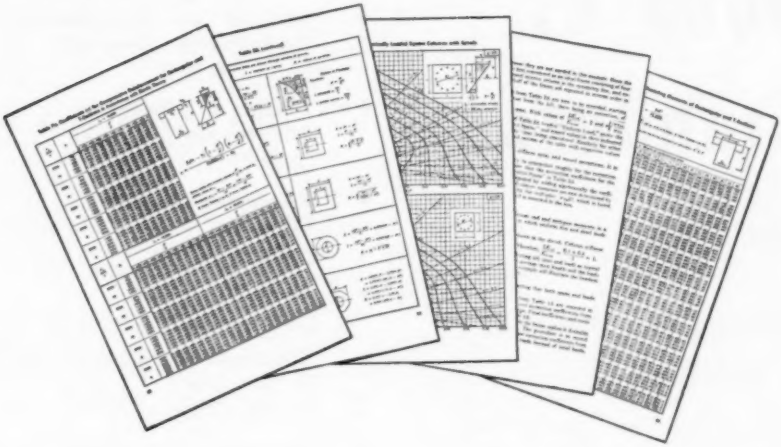

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ENGINEERING SOCIETIES DIRECTORY—1959 edition offers comprehensive United States listing of engineering and scientific societies, their functional staff personnel, and publications. Lists national and international organizations and all state registration boards for professional licensing. A nearly complete listing of Canadian engineering organizations is also included. $3.50 per copy.—Engineers Joint Council, 29 West 39th St., New York 18, N. Y.

CROSS COAL AND AGGREGATE STEEL SCREENS—How to specify perforated metal screens is described in 12 steps. Brochure describes Cross Hexcreen, Conidure pierced sheets, and Rima wedge-shaped-wire screens.—Cross Perforated Metals Plant, National-Standard Co., Carbondale, Pa.

BACK IN THE BLACK (Bulletin 659)—Describes paving, patching, and grouting materials and brick, all emery-based, and bonding and curing agents for floors. Emery aggregate is described.—Walter Maguire Co., Inc., 60 East 42nd St., New York 17, N. Y.

SOUND TRANSMISSION LOSS THROUGH CONCRETE MASONRY WALLS—Lightweight Concrete Information Sheet No. 8 discusses efficient and economical control of sound. Discussion is concerned primarily with the suppression of sound originating outside the room by evaluating the transmission loss through concrete masonry partitions. Two charts and sound intensity table augment material.—Expanded Shale Clay and Slate Institute, National Press Bldg., Washington, D. C.

CONSIDERATIONS FOR CONCRETE AND MORTAR (Bulletin X-6)—Check list of vital factors to be considered when specifying concrete and mortar. —The Master Builders Co., Cleveland 3, Ohio

NON-DESTRUCTIVE TESTING—Bulletin describes the Sonometer apparatus used in nondestructive testing of solid materials by induced sonic vibrations. Features pictures of the apparatus, a block diagram of a typical application, and several examples of the manner in which tests can be performed.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

ENGINEERING CONSTRUCTION BIDDING PROCEDURE GUIDE—Developed by the American Society of Civil Engineers-Associated General Contractors of American joint cooperative committee.

BUILDING CONSTRUCTION BIDDING PROCEDURE GUIDE—This is the third major revision of this guide which was developed in 1948 through the cooperation of the committee on contract documents of the American Institute of Architects and special committee of the AGC.

CHECK LIST FOR INSURANCE AND BONDS—The insurance section is a revised version of earlier AGC publication, and the bond part was developed recently in cooperation with the Surety Association of America. Individual copies of these documents available without charge.—Associated General Contractors of America, Inc., 20th and E St., N. W., Washington 6, D. C.

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